California High-Speed Train Project

Request for Proposal for Design-Build Services

RFP No.: HSR 11-16
Geotechnical Data Report
Clinton Ave to East American Ave

Prepared for

CALIFORNIA High-Speed Rail Authority

CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

FINAL

Fresno to Bakersfield

Geotechnical Data Report
Contract Package 1

February 2012

CALIFORNIA
High-Speed Rail Authority

Fresno to Bakersfield

Geotechnical Data Report
Contract Package 1

Prepared by:

URS/HMM/Arup Joint Venture

February 2012

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ABBREVIATIONS

ASTM	ASTM International (formerly American Society for Testing and Materials)
Authority	California High-Speed Rail Authority
BGS	Below Ground Surface
Caltrans	California Department of Transportation
CDWR	California Department of Water Resources
CEQA	California Environmental Quality Act
cm	centimeter
CPT	Cone Penetration Test
EL	Elevation
EPA	US Environmental Protection Agency
F-B	Fresno to Bakersfield
ft	feet
GBR-B	Geotechnical Baseline Report for Bid
GDR	Geotechnical Data Report
GI	Ground Investigation
HMM	Hatch Mott MacDonald
HST	California High-Speed Train
ICC	International Code Council
JV	HMM/URS/Arup Joint Venture
L	liter
m	meter
mg	milligram
mi	miles
mm	millimeters
MCE	Maximum Considered Earthquake
MSL	Above Mean Sea Level
NA	Not Available
NAD83	1983 North American Datum
NAVD88	1988 North American Vertical Datum
NEHRP	National Earthquake Hazards Reduction Program
NTD	Notice to Designers
OBE	Operating Basis Earthquake
CP1	Contract Package 1
PMT	California High-Speed Train Project Management Team
PPDT	Pore Pressure Dissipation Test
ppm	parts per million
PS logging	P- and S-wave suspension velocity logging
RTW	Rothberg, Tamburini & Winsor, Inc.
SBT	CPT Soil Behavior Type
SBT_N	Normalized CPT Soil Behavior Type
SCPT	Seismic Cone Penetration Test
SJV	San Joaquin Valley
SM	Standard Methods for the Examination of Water and Wastewater
SPT	Standard Penetration Test
SR	State Route
T	Period
TM	Technical Memorandum
umhos	micromhos
USCS	Unified Soil Classification System
USGS	United States Geological Survey
V_{s30}	Average Shear Wave Velocity in the upper 30 meters of soil

1.0 Executive Summary

The California High-Speed Train (HST) Project will provide intercity, high-speed train service throughout California's major population centers. A joint venture (JV) between URS, Hatch Mott MacDonald (HMM), and Arup has been contracted by the California High-Speed Rail Authority (Authority) to perform 30% design–level engineering services for the portion of the project that extends between Fresno and Bakersfield.

This Geotechnical Data Report presents findings from the ground investigation (GI) performed for the 30% design level pertaining to the subsection of the alignment within the city of Fresno metropolitan area, Contract Package 1 (CP1).

The entirety of CP1 extends from Veterans Boulevard to E American Avenue. The northern section of CP1, from Veterans Boulevard to W Clinton Avenue is within the Merced to Fresno segment of the HST. The southern segment of CP1, from W Clinton Avenue to E American Avenue, is within the Fresno to Bakersfield (F-B) segment of the HST. The JV is responsible for this segment. For brevity, where CP1 is referred to in this report, it shall be construed to mean only the F-B section of the corridor contracted to the JV.

The CP1 corridor spans approximately 9 miles from W Clinton Avenue to about E American Avenue. The proposed alignment includes a combination of at-grade sections, grade separations, overcrossings, undercrossings, and aerial structures. The design requires shallow and deep foundations, excavations on the order of 55 feet, retaining walls, and earthwork embankments for the proposed improvements.

The purpose of this report is to present the geotechnical data collected to date (including studies from the 15% design phase) and to support the JV design team in future studies of the proposed improvements.

The ground investigation for CP1 was performed in general conformance with Technical Memorandum (TM) 2.9.1 Geotechnical Investigation Guidelines (Rev 1, 03 Jun 11), TM 2.9.2 Geotechnical Reports Preparation Guidelines (Rev 1, 03 June 11), and Notice to Designers (NTD) No. 001 (April 16, 2010).

The investigation was conducted between October 10 and 28, 2011, and consisted of 17 rotary-wash boreholes and 44 Cone Penetration Tests (CPTs). In general, soil samples were obtained at 5-foot intervals to the bottom of each borehole using Standard Penetration Test (SPT) samplers. In selected boreholes, continuous sampling was performed to target particular depths of interest. All samples were classified in accordance with the Unified Soil Classification System (USCS).

At the completion of drilling, seven boreholes were converted to standpipe piezometers to monitor seasonal groundwater fluctuations; the remaining boreholes and all CPTs were backfilled with neat cement grout, in accordance with local permitting agency regulations.

Laboratory testing was performed on representative soil samples to obtain index and engineering properties. Geotechnical index testing included moisture content, No. 200 sieve wash, hydrometer, grain-size analysis, Atterberg limit, and organic content tests. Engineering property tests included remolded direct shear, compaction, California Bearing Ratio, and corrosion tests.

In situ testing performed during the exploration program included shear wave velocity measurements in four boreholes and six CPTs, and pore water pressure dissipation tests in 19 CPTs.

2.0 Introduction

2.1 Project Description

The California HST Project will provide intercity, high-speed train service on over 800 miles of tracks throughout California, connecting the major population centers of Sacramento, the San Francisco Bay Area, the Central Valley, Los Angeles, the Inland Empire, Orange County, and San Diego. The HST system is envisioned as a state-of-the-art, electrically powered, high-speed, steel-wheel-on-steel-rail technology, including state-of-the-art safety, signaling, and automated train-control systems. The trains will be capable of operating at speeds of up to 220 miles per hour over a fully grade-separated alignment, with an expected express trip time between Los Angeles and San Francisco of 2 hours and 40 minutes.

The entirety of CP1 extends from Veterans Boulevard to E American Avenue. The northern section of CP1, from Veterans Boulevard to W Clinton Avenue is within the Merced to Fresno segment of the HST. The southern segment of CP1, from W Clinton Avenue to about E American Avenue, is within the F-B segment of the HST. The JV is responsible for this segment. For brevity, where CP1 is referred to in this report, it shall be construed to mean only the F-B section of the corridor contracted to the JV.

The F-B CP1 project alignment starts at the intersection of W Clinton Avenue and N Golden State Boulevard in Fresno, California. The alignment continues southeast along Golden State Boulevard for about 2 miles to W Belmont Avenue. South of W Belmont Avenue, the alignment continues southeast between an existing rail right-of-way and G Street for about 2.8 miles where G Street terminates at Golden State Boulevard. At this point, the alignment continues southeast about 1.1 miles between the existing rail right-of-way and Golden State Boulevard until its intersection with E Jensen Avenue, where it veers south crossing Golden State Boulevard, E North Avenue, Golden State Freeway, and E Central Avenue. South of E Central Avenue, the alignment is adjacent to an existing rail right-of-way and continues south for about 2 miles until the end of CP1 at E American Avenue.

The alignment is adjacent to Roeding Park north of W Belmont Avenue; crosses the Sequoia Kings Canyon Freeway (State Route 180), Yosemite Freeway (SR 41), and Golden State Highway (SR 99); crosses irrigation canals north of SR 180 and south of E Central Avenue; and is adjacent to detention basins at the intersection of E McKinley Avenue and N Weber Avenue, and the intersection of the existing rail right-of-way and W Belmont Avenue.

The CP1 alignment includes at-grade and embankments rail sections, a grade separation, a trench, a viaduct, and a jacked-box tunnel. This contract also includes numerous secondary transverse vehicular and pedestrian bridges at select local street intersections. The design requires shallow and deep foundations, retaining walls, and earthwork embankments for the proposed improvements.

The key project features are described in Table 2.1-1, from north to south. A site vicinity map showing the CP1 study area is presented in Figure 2.1-1.

Table 2.1-1
Summary of Significant Structures in Contract Package 1

Name	Approximate Start/End Station (ft)	Design Phase	Physical Location	Approximate Size	Notes
At-grade	10806+00 to 10885+00	30%	Along N Golden State Blvd, from W Clinton Ave to 1,500 ft south of W Olive Ave adjacent to Roeding Park	Width: 60 ft Length: 8,500 ft	At-grade alignment is adjacent to detention basins at W McKinley Ave. Other planned improvements include vehicular overcrossings at W McKinley and W Olive Ave.
Fresno Grade Separation	10885+00 to 10970+00	30%	From Roeding Park to a point about 1,200 ft southeast of El Dorado St	Width: 60 ft Depth: 55 ft Length: 7,400 ft	The Fresno Grade Separation is adjacent to the Belmont Detention Basin. It crosses under W Belmont Ave. SJVRR spur, Dry Creek Canal, SR 180. Jacked Box tunnel is proposed where alignment crosses SR 180.
At-grade	10970+00 to 11300+00 (257+24.66 to 352+25)	30% (15%)[1]	From point about 1,200 ft southeast of El Dorado St to about a point about 400 ft southeast of E Church Ave	Width: 100+ ft Length: 12,500 ft	HST overcrossings are proposed at Fresno St, Tulare St, and Ventura Ave. Pedestrian bridges between Tuolumne St and Stanislaus St, Ventura Ave, and E Church Ave. Vehicular overcrossings Stanislaus St, Tuolumne St, Ventura Ave, and E Church Ave. The Fresno Station is planned in this reach of the alignment.
Jensen Trench	352+25 to 421+25	15%	From a point about 400 ft southeast of E Church Ave to S Orange Ave	Width: 100 ft Depth: 17 ft Length: 4,400 ft	Crosses under E Jensen Ave
Fresno Viaduct	439+48 to 505+91.35	15%	From Golden State Blvd to a point about 500 ft north of E Muscat Ave	Width: 60 ft Height: 40 ft Length: 5,500 ft	Crosses Golden State Blvd, E North Ave, S Cedar Ave, and SR 99.
Embankment/ At-Grade	505+91.35 to 627+08.33	15%	From a point 500 ft north of E Muscat Ave to E Lincoln Ave	Width: 60 ft Length: 13,400 ft	Crosses irrigation canals adjacent to E Central Ave, between E Malaga and E American Ave, and between E Jefferson and E Lincoln Ave. Vehicular overcrossings are proposed at E American Ave and E Central Ave crossings.

[1] Structure begins in Package 1A/1B (30% design) and ends in Package 1C (15% design)

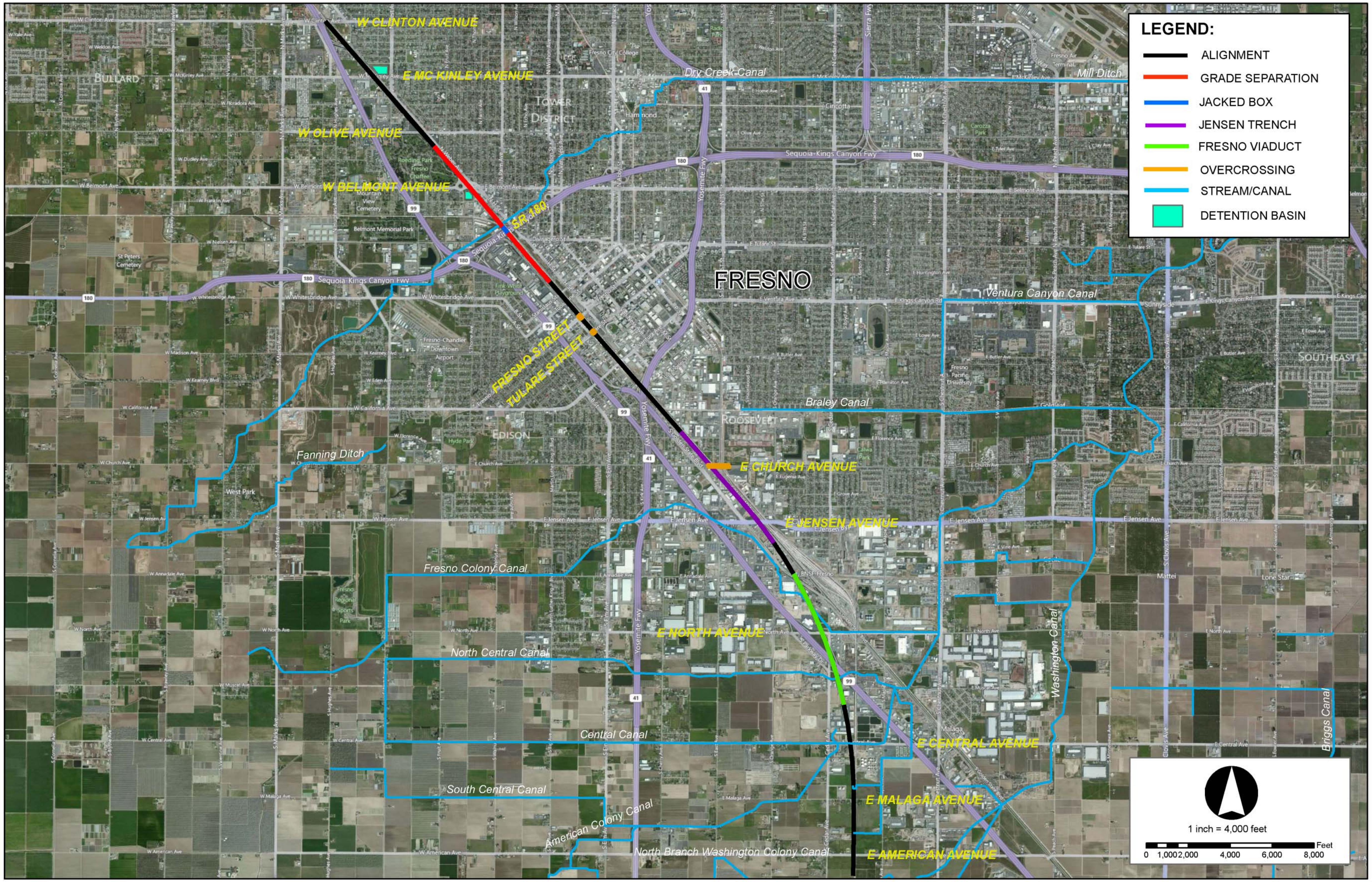

Figure 2.1-1
Site Vicinity Map

2.2 Purpose and Scope

2.2.1 Purpose

The purpose of the 30% F-B CP1 Geotechnical Data Report (GDR) is to present the geotechnical data collected to date (including studies from 15% design phase) and to support the JV design team in future studies of the proposed improvements.

This report supports 30% design geotechnical memoranda and design reports for each area of interest along the alignments and the Geotechnical Baseline Report for Bid (GBR-B). The memoranda and reports will provide interpretation of the data presented in this report to satisfy the following specific goals:

- Partially meet the requirements of recommended mitigating measures in the F-B Environmental Impact Report/Statement
- Define the lateral and vertical variability in subsurface soil conditions for 30% design-engineering efforts and construction cost estimates
- Confirm the depth, seasonal, and spatial variability of groundwater for 30% design and construction cost estimates
- Assess potential hazards identified in the F-B Geologic and Seismic Hazards Report (URS/HMM/Arup 2011c) and provide mitigation recommendations
- Determine engineering design parameters for preliminary 30% design geotechnical analyses of structures — including viaducts, bridges, stations, trenches, local road crossings, embankments, and retaining walls — to validate the feasibility of the alignment
- Report the data and analyses in GDR and GBR-B as specified in the JV's Fiscal Year 2011/2012 Annual Work Plan, under Task 4.11.11 Deliverables

2.2.2 Scope

A GI was performed to obtain field and laboratory testing information on subsurface conditions to support the 30% design. Supplementary surface and subsurface investigations are required to support the design-bid-build and final designs. In order to satisfy the project requirements, the subsurface investigation included the following:

- Boreholes at 17 locations to define the subsurface stratigraphy, carry out in situ testing, and collect soil samples for visual classification and laboratory testing
- Installation of 7 piezometers to monitor groundwater-level fluctuation
- CPTs at 44 locations to supplement the boreholes and help define the subsurface stratigraphy and develop parameters for engineering analysis and design
- Downhole P- and S-wave suspension velocity logging (PS logging) at 4 locations to confirm the soil site class and develop parameters for seismic design
- Laboratory testing to characterize the major strata and develop parameters for engineering analysis and design

A detailed GI specification was developed to prequalify GI and laboratory testing contractors for pricing. Gregg Drilling and Testing, Inc. (Gregg Drilling) was selected as the recommended subcontractor to perform the 30% F-B GI work for the JV with Authority approval. Laboratory testing was procured under a separate contract. Sierra Testing Laboratories, Inc. was selected to perform the geotechnical laboratory testing.

2.3 Available Data and Information

2.3.1 Project Sources

Available data and information for this report include data collected through desk studies available on ProjectSolve, geotechnical reports prepared by the JV, and TMs prepared by the HST Project Management Team (PMT). Geotechnical reports previously prepared by the JV include the following:

- F-P Geotechnical Data Report – Historical Borehole Data (URS/HMM/Arup 2010a)
- F-B Draft Environmental Impact Report/Environmental Impact Statement (URS/HMM/Arup 2011a)
- F-B Geology, Soils, and Seismicity Report (URS/HMM/Arup 2011b)
- F-B Geologic and Seismic Hazards Report (URS/HMM/Arup 2012a)
- F-B Geotechnical Investigation Work Plan (URS/HMM/Arup 2012b)
- F-B Water Quality/Hydrology Report (URS/HMM/Arup 2012c)

Available TMs, NTDs, and other information from the PMT related to geotechnical and geological investigations, and geotechnical engineering pertinent to the preparation of this report include the following:

- TM 2.9.1 Geotechnical Investigation Guidelines (PMT 2011a)
- TM 2.9.2 Geotechnical Reports Preparation Guidelines (PMT 2011b)
- TM 2.9.3 Geologic and Seismic Hazard Analysis Guidelines (PMT 2011c)
- TM 2.9.10 Geotechnical Design Guidelines (PMT 2011d)
- TM 2.10.4 Seismic Design Criteria (PMT 2011e)
- NTD No. 01 – Geotechnical Investigations for Preliminary Design, R1 (PMT 2011f)
- NTD No. 03 – Preliminary Engineering (30% Design) Scope Revisions, R0 (PMT 2011g)
- NTD No. 08 – Geotechnical Boring and Sample Identification, Handling and Storage Guidelines R0 (PMT 2011h)
- Interim 30% Design Spectra for F-B (PMT 2011i)

2.3.2 Information from Other Sources

Available geotechnical data from historical projects near the study area were collected as part of 15% geotechnical design efforts.

The primary source of publically available geotechnical data was from California Department of Transportation (Caltrans) collection of as-built construction records. Caltrans data is concentrated along SR 41, SR 43, and SR 99, from projects dating between 1953 and 1997. For each project, several boreholes were drilled, logged, and plotted on a cross section. None of the Caltrans records contain laboratory test data.

Borehole records collected from Caltrans extend to a maximum depth of 121.8 feet below ground surface (BGS).

All relevant data from these reports have been included as an attachment to this report in Appendix A.

2.4 Report Structure

Section 1 provides an Executive Summary for the report, while Section 2 provides an introduction to the project including a project description, report purpose and organization, and a summary of available data and information. Sections 3, 4, and 5 describe the project setting through geology, seismicity, and hydrogeology, respectively. The GI program is described in Section 6, and the results of the laboratory testing program are summarized in Section 7. The report closes with a discussion of surface and subsurface conditions in Section 8 and limitations in Section 9. References are provided in Section 10.

3.0 Physiography and Geologic Setting

The section provides a brief description of the project physiography, geologic deposits, and seismicity. A detailed seismicity evaluation is presented in the Geologic Seismic and Hazards Report (URS/HMM/Arup 2012c).

3.1 Physiography

The California HST F-B alignment is located in the south portion of the Great Valley Geomorphic Valley (commonly referred to as the San Joaquin Valley [SJV]). The topography of the Great Valley is relatively flat; it is bordered by the Pacific Coast Range to the west, the Klamath Mountains and Cascade Range to the north, the Sierra Nevada to the east, and San Emigdio and Tehachapi mountains to the south.

Superimposed upon this large-scale, relatively flat topography is a localized topography caused by recent incisions of river systems. The subsequent topography comprises short, steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) and narrow gully-type valleys, depending on their age and the amount of flow; however, along the CP1 alignment, these features appear to have been either channelized or redirected along more convenient routes to accommodate the present urbanization.

The topography along the CP1 corridor is generally flat and varies between elevation (EL) 285 and 295 feet above mean sea level (MSL). Localized variations on the ground surface elevation occur at existing road embankments, detention basins, and other man-made features such as irrigation canals and road crossings.

3.2 Geologic Setting

3.2.1 Regional Geology

The SJV comprises the southern part of the 450-mile-long Great Valley of California. It is an asymmetric structural trough that is filled with prism sediments up to 30,000 feet thick. It formed the southern part of an extensive fore-arc basin that evolved during the Cenozoic into today’s hybrid intermontane basin.

The SJV evolved through the gradual restriction of the marine basin due to uplift and emergence of the northern Great Valley in the late Paleogene, the closing off of the western outlets in the Neogene, and finally the sedimentary infilling in the Neogene and Quaternary. These sediments rest on crystalline basement rocks of the southwestward-tilted Sierran block.

Figure 3.2-1 shows a cross-sectional schematic of the Great Valley deposits.

3.2.2 Local Geology

The local geology of the Fresno area is created by the low alluvial fans of the perennial San Joaquin River and four ephemeral streams that form the Alluvial Fan sequence. The Pleistocene formations that make up the Fresno fan sequence are the Modesto (Qf), Riverbank (Qc), and Turlock (Qp) formations. These deposits make up the major surface and subsurface units and originate from stream channels emanating from the foothills east of Fresno. They are similar in mineralogy, deposition, and source.

The Modesto formation occupies the highest stratigraphic position. Sediments within the Modesto formation range in grain size from clay to gravel and seldom exhibit well-developed sedimentary structures. The Riverbank formation underlies the Modesto formation, but does not differ greatly

in lithology or texture. It is also characterized by the occurrence of a laterally extensive, but not pervasive, hardpan member.

The Turlock formation is the oldest unit exposed in the Fresno alluvial fan sequence and forms extensive subsurface deposits throughout the SJV. It contains the majority of the hydrologically important subsurface deposits in the Fresno area. However, it is unlikely this unit will be encountered during geotechnical studies for the HST project due to its depth.

Six well-defined clay layers (designated “A” through “F”), underlie the central part of the SJV. The Corcoran clay layer (clay layer “E”) is the most well-known and extensive of these layers. This layer underlies the city of Fresno at a depth of about 300 feet below existing grade (Brown & Caldwell 2006) as shown in Figure 3.2-2.

South of E North Avenue there is a possibility of encountering sand dunes overlying the Modesto Formation. Aeolian sand dunes appear on some geologic maps but not others. Figure 3.2-3 shows the approximate extent of the sand dunes (Page 1986).The sand dunes have been described to have a relief of about 5 to 20 feet and are associated with a group of surface depressions that trend southeast (CH2M Hill 2005). These dune deposits are well sorted (poorly graded) and moderately permeable.

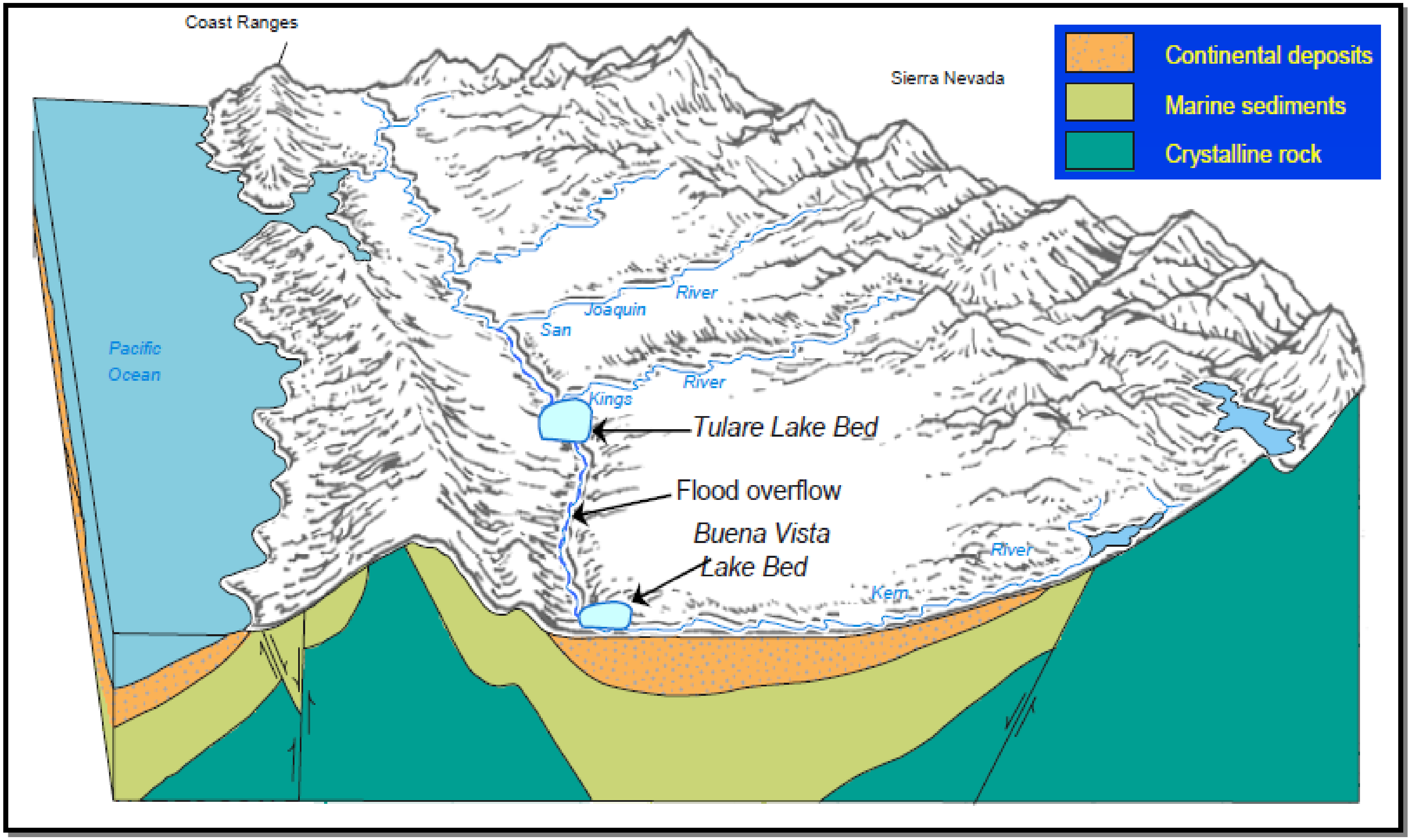

Figure 3.2-1
Cross Section of Great Valley Geomorphic Province (Page 1986)

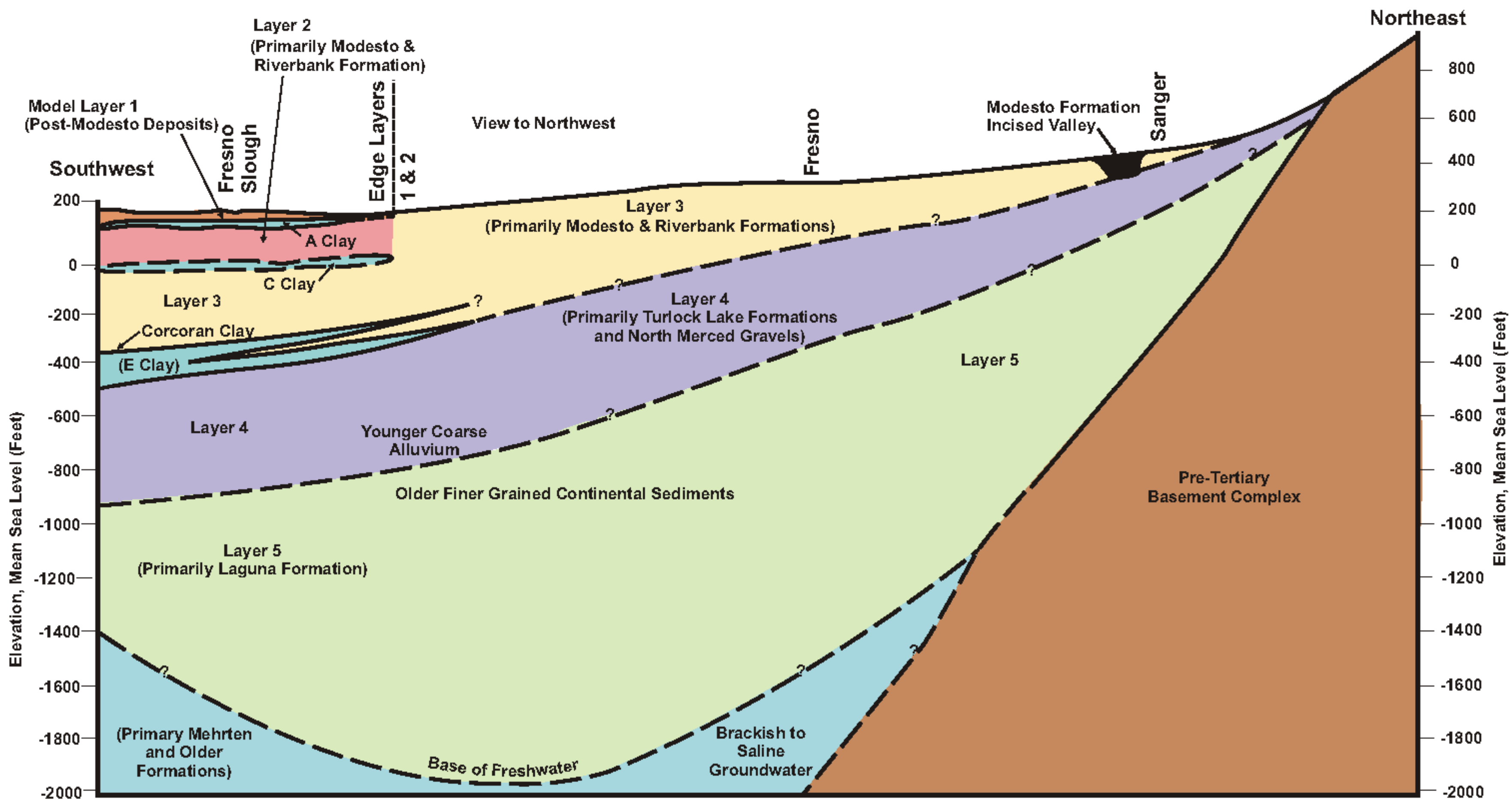

Figure 3.2-2
Geologic and Hydrogeologic Cross Section (Brown and Caldwell 2006)

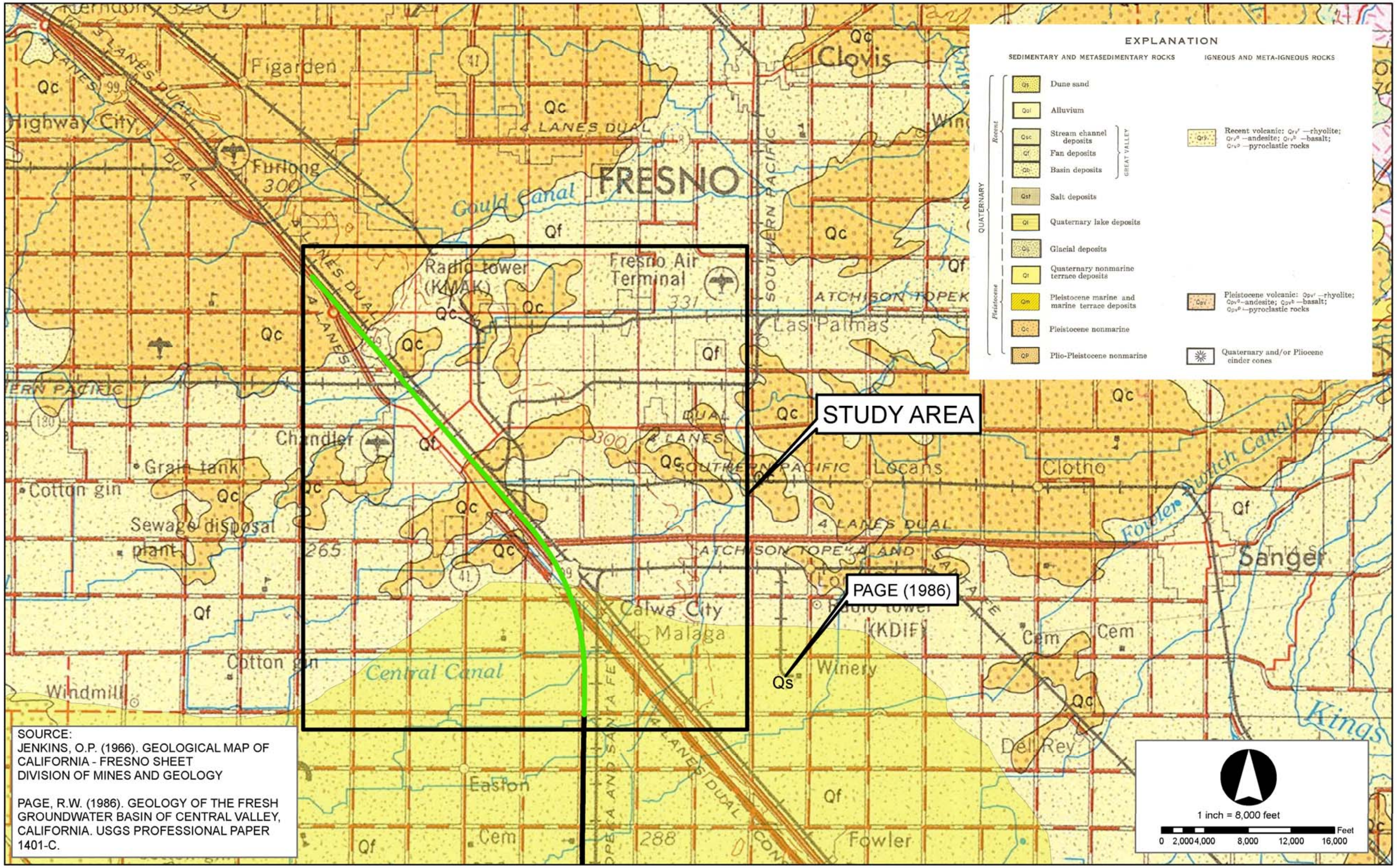

Figure 3.2-3
Local Geology in the Study Area (Jenkins 1966 and Page 1986)

4.0 Seismic Setting

4.1 General Seismic Setting

The study area is located within a relatively seismically quiescent region between two areas of documented tectonic activity: the Coast Ranges-Sierran Block boundary zone to the east and the Pacific Coast Ranges boundary zone to the west.

The Coast Ranges-Sierran Block, which follows the physiographic boundary between the Coast Ranges and Great Valley geomorphic provinces, contains potentially active blind thrust faults (Unruh and Moores 1992). Based on the size of historical events and on the inferred subsection of the boundary zone, these blind thrust faults are capable of producing moderate to large earthquakes. The Pacific Coast Ranges contain many active faults that are associated with the northwest-trending San Andreas Fault System (Jennings 1994), which is the principal tectonic element of the North American/Pacific plate boundary in California.

In the SJV, seismic slip is partitioned onto subsidiary structures, such as the San Andreas, Garlock, and Coalinga Faults, which are distributed across the Great Valley geomorphic province but not in close proximity to the study area.

4.2 Capable Faults

There are no known active faults crossing or within close proximity to the alignment within the study area. The San Andreas Fault, located approximately 65 miles from the site, has the highest slip rate and is the most seismically active of any fault near the HST alignment. The closest fault to the alignment is the Clovis Fault; however, the potential seismicity of this fault has not been characterized in the literature reviewed.

There are a number of other faults capable of producing large magnitude earthquakes near the HST alignment. A list of known faults within 100 miles of the study area and their characteristics is presented in Table 4.2-1. These faults are shown in Figure 4.2-1 along with other mapped quaternary faults in the vicinity of the study area.

Table 4.2-1
Characteristics of Capable Faults within 100 miles of the Study Area (USGS 2006)

Fault Name	Fault Type	Moment Magnitude (M_w)	Slip Rate (mm/yr)	Distance and Bearing to Alignment
San Andreas	RL/SS	7.4	20 to 35	65 miles southwest
Great Valley (Segments 10–14)[1]	BT	6.5	1.5	50 miles southwest
Ortigalita	RL/SS	7.1	0.5 to 1.5	65 miles west
San Joaquin	R	6.9	-	57 miles west
O'Neill	R	6.7	-	58 miles west
Nunez	-	-	-	53 miles southwest
Foothills	N	6.5	0.1	90 miles northwest
Round Valley/Hilton Creek	N	7.0/6.7	1	80 miles northeast
Clovis[2]	-	-	-	12 miles east

[1] Caltrans (1996)
[2] California Geological Survey (2010)
N = normal, BT = blind thrust, R = reverse, RL = right lateral, SS = strike-slip

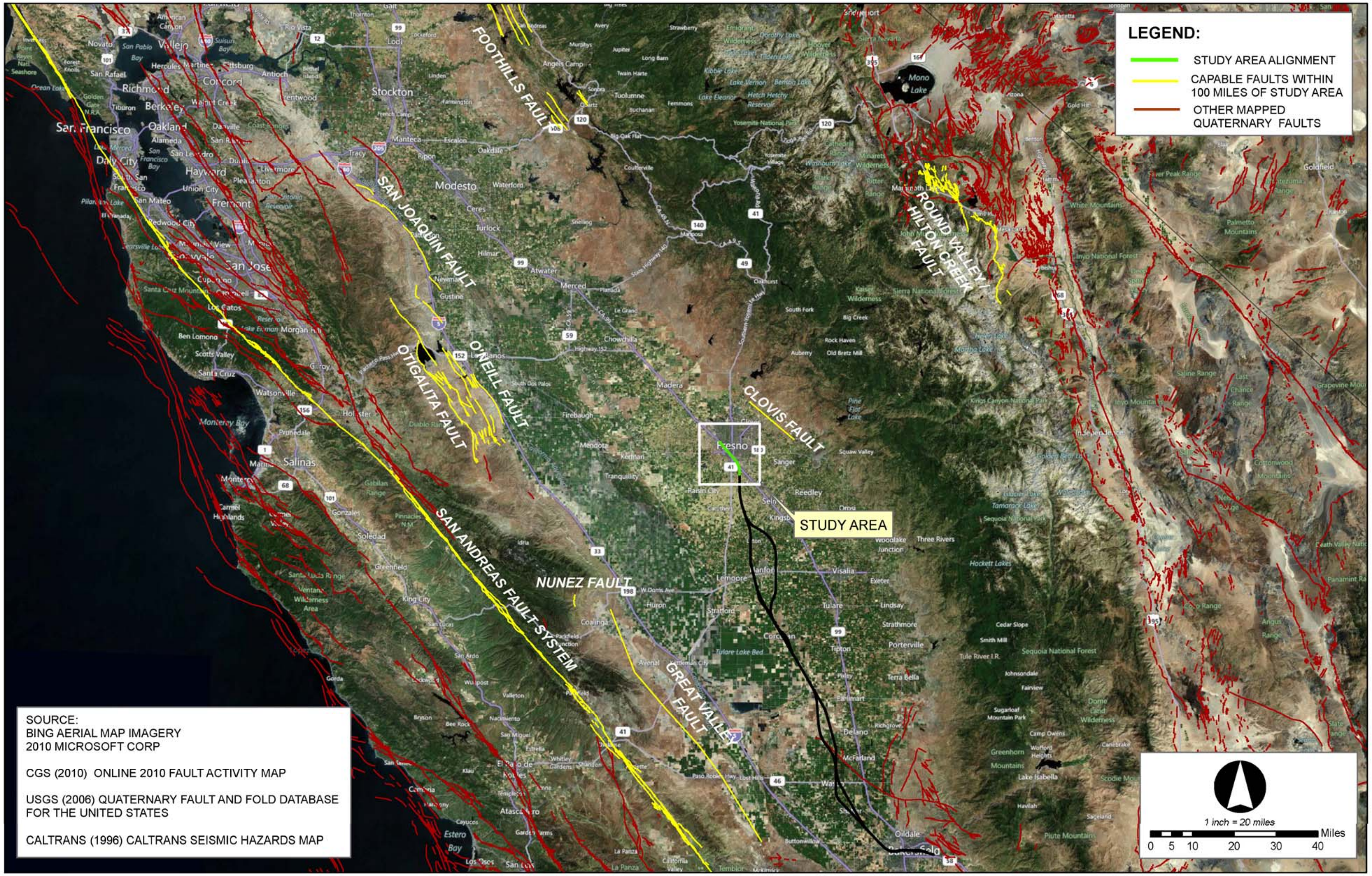

Figure 4.2-1
Mapped Faults in Vicinity of Study Area

4.3 National Earthquake Hazards Reduction Program Site Class

When developing the seismic design ground motions discussed below, the PMT assumed an average shear wave velocity (V_{s30}) of 935 feet per second prior to any site-specific GI. Based on Table 4.3-1, this corresponds to the median shear wave velocity for Site Class D.

Table 4.3-1
National Earthquake Hazards Reduction Program Site Class Definitions (ICC 2006)

Site Class[1]	Soil Profile Name	Average Properties in Upper 100 ft (~30 m) Shear Wave Velocity, V_{s30}	
		ft/sec	m/sec
A	Hard rock	$V_{s30} > 5{,}000$	$V_{s30} > 1{,}524$
B	Rock	$2{,}500 < V_{s30} \leq 5{,}000$	$762 < V_{s30} \leq 1{,}524$
C	Very dense soil and soft rock	$1{,}200 < V_{s30} \leq 2{,}500$	$366 < V_{s30} \leq 762$
D	Stiff soil profile	$600 < V_{s30} \leq 1{,}200$	$183 < V_{s30} \leq 366$
E	Soft soil profile	$V_{s30} < 600$	$V_{s30} < 183$

[1] As defined in 2006 International Building Code Section 1613.5.5 (ICC 2006)

4.4 Seismic Design Criteria

The system performance criteria approach uses design earthquakes to which HST facilities are to be designed. As more devastating earthquakes have a lower probability of occurrence, design engineers frequently use a probabilistic approach to define earthquake hazard level. A "return period" identifies the expected rate of exceedance of a given ground motion level. In certain cases, the PMT used deterministic methods to evaluate singular, severe earthquake scenarios based on faulting, site-to-source distance, and geologic conditions.

4.4.1 Design Earthquakes

For the Fresno portion of the HST alignment, two design-level earthquakes are defined as follows in accordance with TM 2.10.4:

Maximum Considered Earthquake (MCE): ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) or (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to maximum moment magnitude) of any fault in the vicinity of the structure.

Operating Basis Earthquake (OBE): ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

4.4.2 Performance Levels

At 30% design, the MCE corresponds to the Non-Collapse Performance Level. The main objective is to limit structural damage to prevent collapse during and after an MCE. The OBE governs

evaluation of the Operability Performance Level. The design objective for the Operability Performance Level is to ensure an elastic response (within structural deformation limits) to the OBE with no spalling.

4.5 Seismic Design Ground Motions

Procedures for defining the seismic design parameters for the HST are defined in TM 2.10.4. The PMT and seconded staff from the Regional Consultant developed site-specific, spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield. The team divided the alignment into eight zones based on shear wave velocities published by the USGS as well as the variations in the calculated ground motion parameters. The CP1 alignment falls within Zone 4 of the PMT-defined study area. Table 4.5-1 summarizes the PMT's seismic design parameters for 30% design.

Table 4.5-1
30% Design Seismic Parameters

Seismic Parameter	OBE	MCE
Peak ground acceleration (g)	0.08	0.25
Moment magnitude	6.7–7.9	7.1–7.9

The PMT also developed smoothed acceleration response spectra for 30% design for Zone 4 (Figure 4.5-1). Figure 4.5-1 shows design response spectra for both vertical and horizontal ground motions. Peak ground accelerations in Table 4.5-1 were taken as the horizontal spectral acceleration at the period (T) of 0.01 seconds.

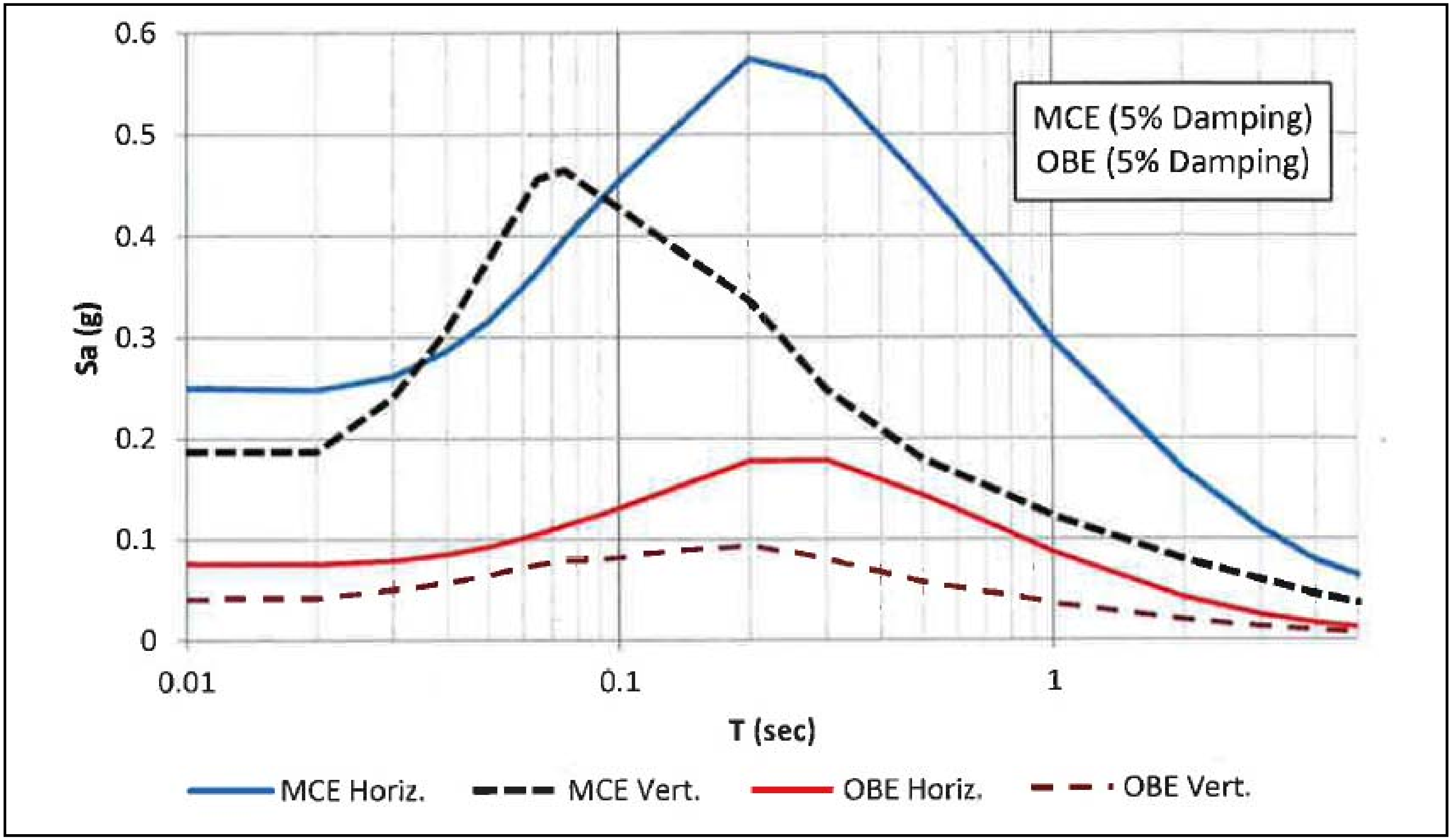

Figure 4.5-1
Design Response Spectra (SC Solutions 2011)

5.0 Hydrogeologic Setting

5.1 Regional Cross Sections

The HST alignment is located within the Kings Sub Basin shown on Figure 5.1-1. A hydrogeologic cross section of the basin is shown on Figure 5.1-1. Groundwater within the northeastern quadrant of the basin is managed under the Fresno Regional Area Groundwater Management Plan. Groundwater is the sole source of drinking water in the region. The current and potential uses of groundwater in the basin are municipal and domestic supply, industrial process supply, industrial service water supply, and agricultural water supply, as specified in the regional management plan (FID et al. 2006).

The regional groundwater flow direction in this area is from east to west. There are some localized influences as a result of pumping, surface water treatment, and groundwater recharge appurtenances. Ponds associated with the Fresno Regional Wastewater Facilities have created a water table high or recharge mound west of Fresno. Historically, shallow groundwater levels have dropped. Regional hydraulic conductivity has been calculated from 1 to 3 feet per day. Local flow rates based on groundwater monitoring data are approximately 1 foot per year (CH2M Hill 2005).

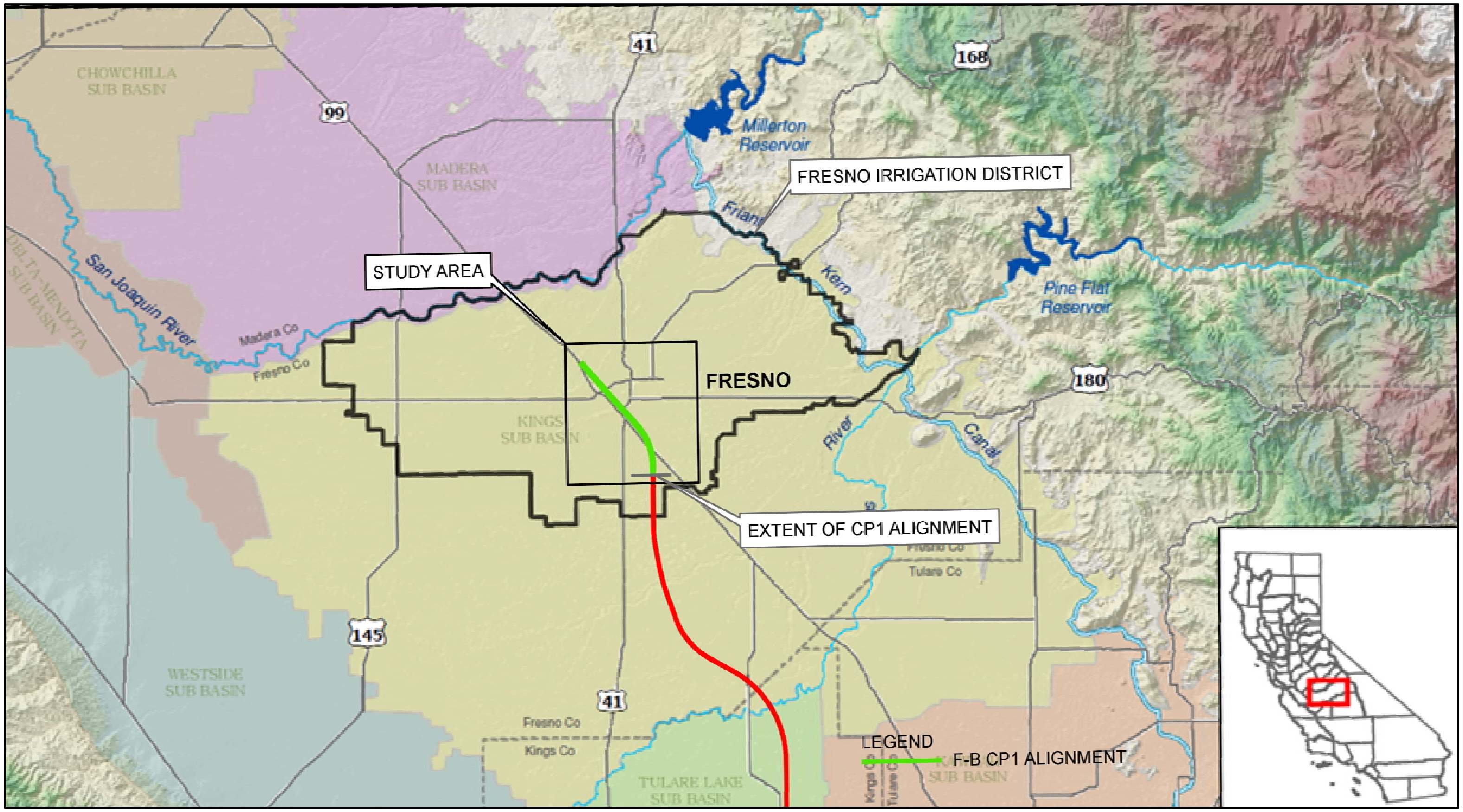

Figure 5.1-1
Kings Groundwater Sub-Basin (FID et al. 2006)

5.2 Major Aquitards

Most of the aquifers underlying the study area are unconfined but may be semiconfined in isolated locations. The primary aquifer in the study area is Fresno Sole Source Aquifer.

Generally, there are no extensive, low-permeability soils that isolate the upper aquifers from the lower aquifers. The Corcoran Clay (E-Clay) underlies the city of Kerman 15 miles to the west but does not extend to the city of Fresno. The Fresno Sanitary Landfill is about 3 miles southwest of the alignment. Three aquifers underlying the landfill (CH2M Hill 2005) have been identified and are described below.

The A-Aquifer is found at approximately 50 to 95 feet BGS. It is mostly fine- to medium-grained, poorly graded sand with interbedded layers of both coarse-grained sands and very fine-grained stiff clayey silts. Regional hydraulic conductivity has been calculated from 1 to 3 feet per day.

Below the A-Aquifer is an aquitard composed of red-brown sandy clay, gray clayey silt, and brown-gray clayey silt. According to the American Geological Institute, an aquitard is a confining bed that retards but does not prevent the flow of water to or from an adjacent aquifer. The B-Aquifer spans from approximately 110 to 150 feet BGS. It is composed of thick interlayers of stiff clayey silt and poorly graded, fine- to medium-grained sand that contains coarse-grained mica flakes. The aquitard below the B-Aquifer is composed of thick clayey silt layers like those in the B-Aquifer.

The C-Aquifer is from approximately 200 to 240 feet BGS. The C-Aquifer is composed of interlayered well- and poorly graded sand and clayey silt. The coarser-grained soils include particles up to large gravel-sized volcanic pumice (pyroclastic) material (CH2M Hill 2005).

5.3 Regional Groundwater Levels

5.3.1 Historical Groundwater Levels

Historically the groundwater table elevation fluctuates but has generally experienced a depletion of about 50 feet since the 1960s. Prior to urbanization and agricultural pumping, the groundwater table was within 20 to 30 feet of the ground surface. Groundwater was not encountered in the majority of the Caltrans boreholes. Figure 5.3-1 shows a hydrograph of historic water well levels in the city of Fresno over the past 80 years. This hydrograph is reasonably consistent with hydrographs of wells along the alignment presented in the F-B Geologic and Seismic Hazard Report (URS/HMM/Arup 2012a) along the HST alignment showing a general trend of groundwater depletion within the Fresno city limits.

Table 5.3-1 summarizes the historical groundwater levels along the alignment over the past 50 years according to various sources at the California Department of Water Resources (CDWR 2011) website, including groundwater wells along the alignment.

Table 5.3-1
Groundwater Table Depths (CDWR 2011)

Location	Existing Grade (MSL, ft)	Time Period				
		1960–65	1984–88	1998–2001	2005	2009–11
W Clinton Ave	298	70	88	98	110	--
Roeding Park	294	64	84	94	100	--
Ventura Ave	291	59	71	80	101	125
E North Ave	287	30	45	67	65	--

5.3.2 Current Groundwater Levels

Urbanization and the pumping demand on the groundwater table have caused a cone of depression within the city of Fresno. Figure 5.3-2 shows regional groundwater elevation contours in the vicinity of the study area in 2005. The figure indicates that the groundwater table was at about EL 180 to EL 190 feet, or about 100 feet below existing grade.

Groundwater levels measured during this investigation indicated similar depths. As of the writing of this report, the groundwater table is at about 90 to 100 feet below existing grade throughout the Fresno city limits and gently rises to about 60 to 70 feet below existing grade toward the southern reaches of CP1. Further discussion of observed groundwater conditions is presented in Section 6.5.

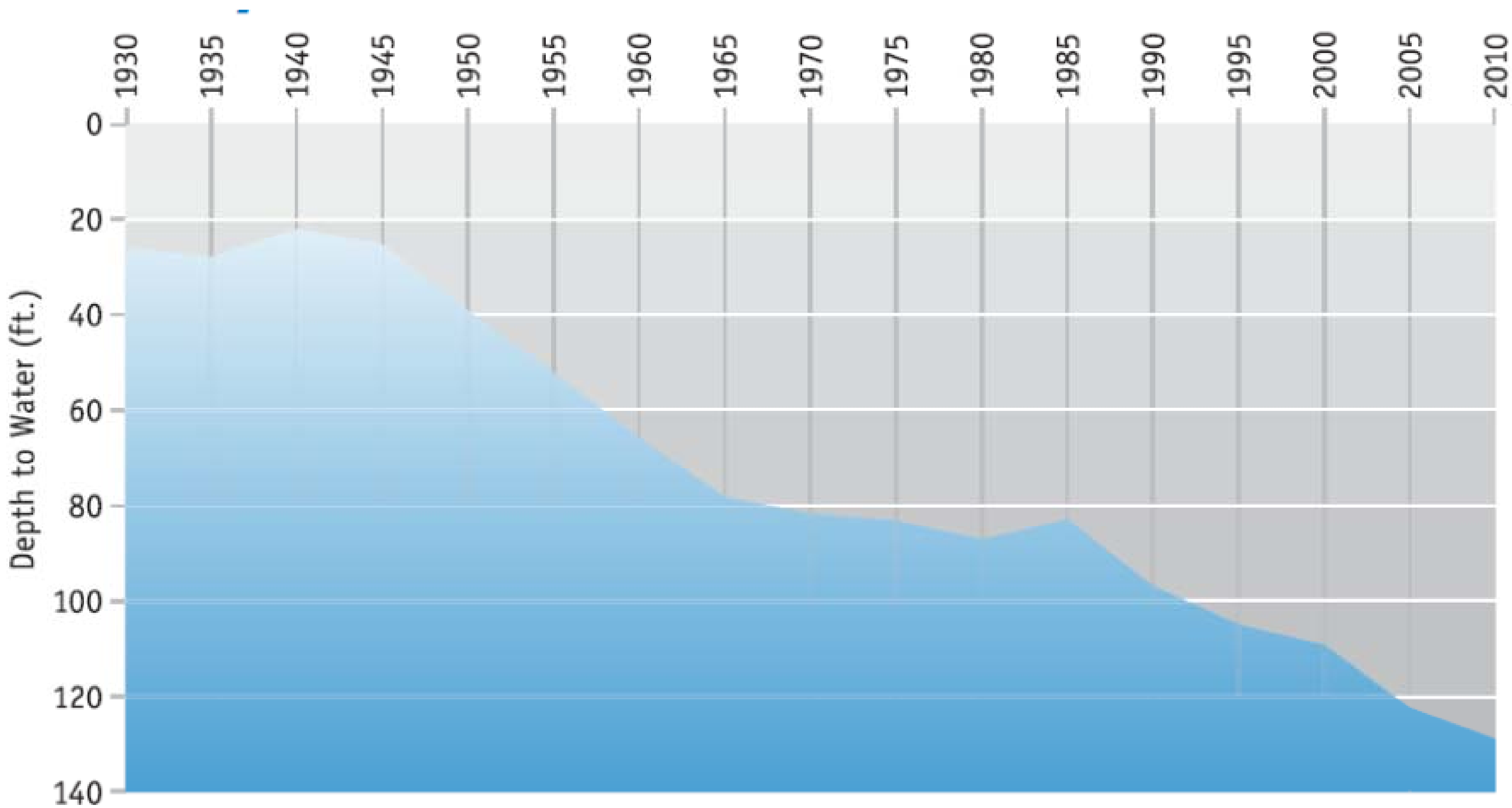

Figure 5.3-1
City of Fresno Depth to Water (City of Fresno 2010)

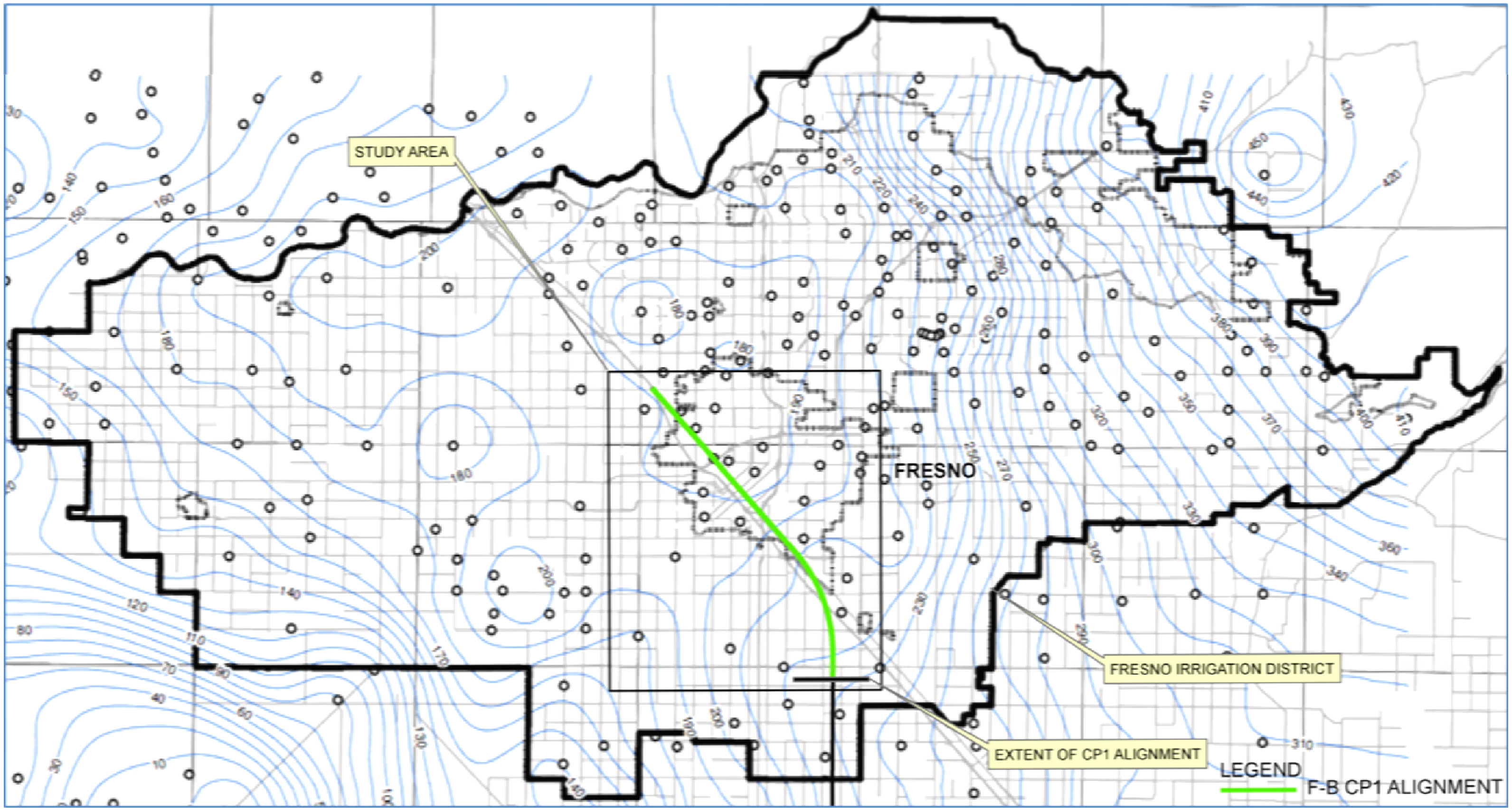

Figure 5.3-2
Groundwater Contours (FID et al. 2006)

5.4 Land Subsidence

No information is available on historic land subsidence within the CP1 study area. The area may have experienced land subsidence in the early 1930s when it was prevalent in the SJV. However, no significant land subsidence is known to have occurred in the last 50 years as a result of land development, water resources development, groundwater pumping, or oil drilling.

A Global Positioning System control network has been established throughout the Fresno Irrigation District (FID). This network consists of more than 20 control points that are tied to the High Precision Grid Network using the North American Vertical Datum 1988 (NAVD88). It is utilized to survey existing local benchmarks to monitor subsidence (FID et al. 2006).

A cursory evaluation of subsidence along the alignment was made by comparison of current ground surface elevations along the alignment to ground surface elevations from Google Earth. Based on this evaluation, there does not appear to be any detectible land subsidence within the CP1 study area.

5.5 Artesian Conditions

While there is some discussion in various references regarding semiconfined aquifers within the study area (USGS 2004), the primary aquifer (Sole Source Aquifer) is considered unconfined.

5.6 Methane Gas Hazard

There are no known sources of methane gas within close proximity to the alignment that would pose a hazard. The Fresno Sanitary Landfill is located approximately 3 miles southwest of the study area. There are no known natural gas fields within the study area.

5.7 Groundwater Chemistry

The groundwater beneath portions of the city of Fresno contains a number of inorganic and organic chemical contaminants, but these are not considered corrosive per se. Two indices are useful to predict the potential for corrosion or scale formation of water:

- **Langlier Saturation Index –** The Langlier Index predicts the scaling of water based on the calcium carbonate equilibrium values. If the actual pH of water is below the calculated pH, the Langlier Index is negative, indicating that the water will dissolve calcium carbonate and that it will be corrosive, particularly if dissolved oxygen is present. If the actual pH of water is higher than the calculated pH, the Langlier Index is positive, indicating that incrustants (i.e., scaling) will likely occur.

- **Ryznar Stability Index** – The Ryznar Index predicts the tendency for scaling and corrosion. It is widely used to predict the reaction of metal in saturated subsurface conditions. Water is corrosive if the index is higher than 7, and incrusting if it is less than 7.

The Rothberg, Tamburini & Winsor, Inc. (RTW) Corrosivity Index Calculator (AWWA 2011) can be used to calculate both the Langlier and Ryznar Indices. Each may be independently used to determine the corrosive nature of a given influent water. Both indices were determined using average groundwater characteristic values obtained from the City of Fresno (2010), and are presented in Table 5.7-1.

Table 5.7-1
Characteristic Groundwater Chemistry Values for Fresno (AWWA 2011)

Initial (Entered) Water Characteristics	
Measured TDS	219 mg/L
Measured Temperature	25°C
Measured pH	8
Measured alk, as CaCO3	114 mg/L
Measured Ca, as CaCO3	114 mg/L
Measured Cl	9 mg/L
Measured SO4	10 mg/L
Theoretical Interim Water Characteristics	
Langlier Index	0.43
Ryznar Stability Index	7.1

6.0 Ground Investigation

6.1 Introduction

The GI for the CP1 study area was performed between October 10 and October 28, 2011. The investigation program included cone penetration testing, exploratory boreholes, downhole geophysical logging, and observation well installations.

The investigation was performed in general conformance with TM 2.9.1, Geotechnical Investigation Guidelines (Rev 1, 03 Jun 11) and TM 2.9.2 Geotechnical Reports Preparation Guidelines (Rev 1, 03 Jun 11), NTD No. 001 (April 16 2010).

A map of exploration locations is presented in Figure 6.1-1.

6.1.1 30% Design

The purpose of the GI was to provide information about the subsurface soil, groundwater, and seismic conditions along the HST Project alignment. Investigations of the subsurface conditions provide geotechnical design parameters to support the JV's design for 30% design engineering. The primary purpose of the 30% design engineering is to verify feasibility and constructability of the proposed standard, nonstandard, and complex structures within the study area.

6.1.2 Organization of Team

The engineer during the field investigation was URS/HMM/Arup JV and the contractor was Gregg Drilling of Signal Hill, California. Gregg Drilling supplied two CPT rigs and two rotary-wash rotary drill rigs for the investigation.

6.1.3 Field Manual

References for field personnel included the following:

- GI Specification (URS/HMM/Arup 2010b)
- GI Work Plan for 30% Design 15% Record Set (URS/HMM/Arup 2011f)
- Health and Safety Plan, for use on the GI Program (URS/HMM/Arup 2011g)

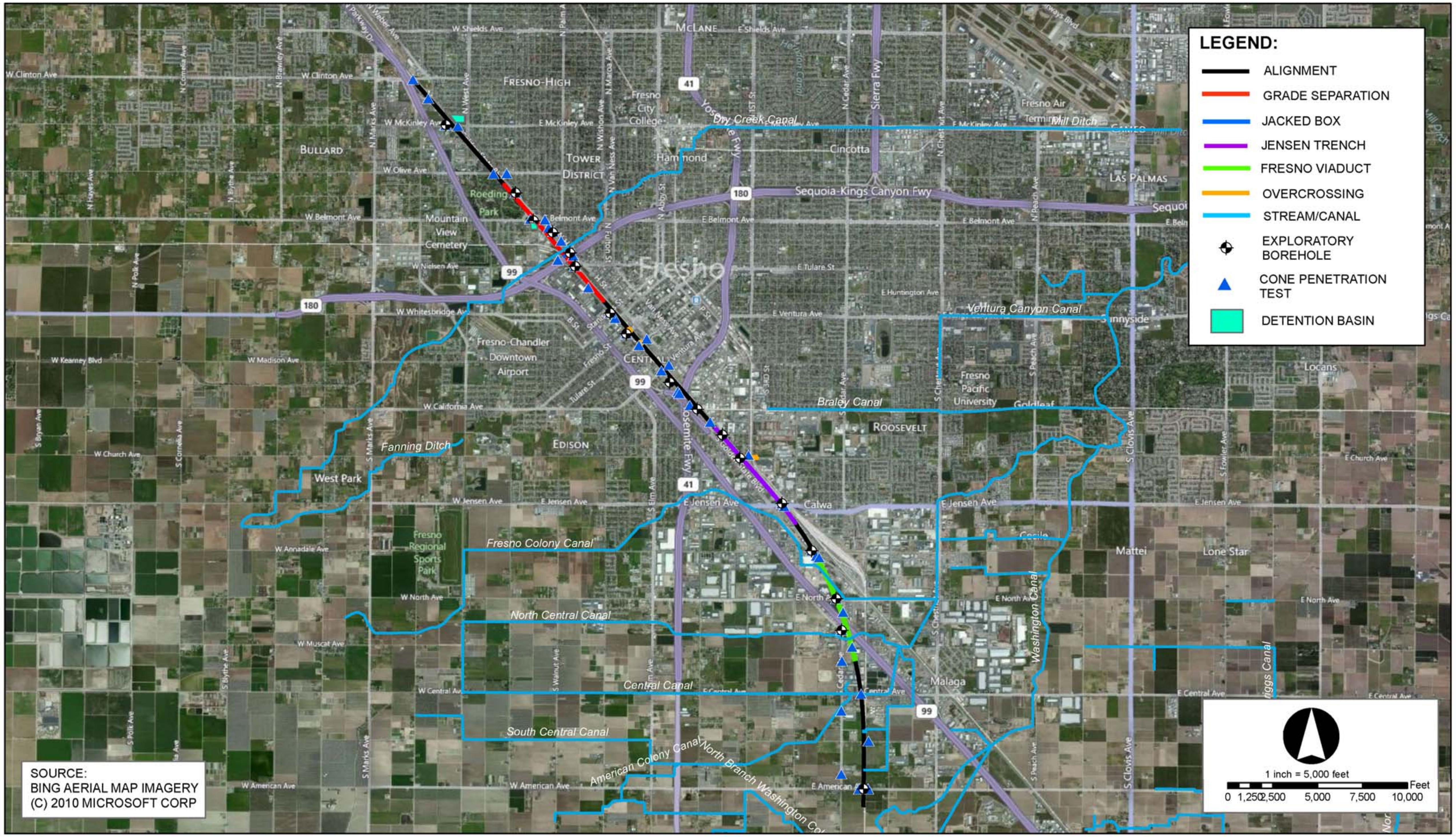

Figure 6.1-1
Map of Exploration Locations

6.1.4 Project Restrictions

A. Local Regulations

The JV contacted the City of Fresno prior to site mobilization to determine permitting fees and requirements for geotechnical drilling, well construction, borehole abandonment, and street encroachment.

In accordance with City regulations, all boreholes and CPTs advanced to within 15 feet of the known depth to groundwater were permitted. At these permitted locations, holes were either abandoned or piezometers were constructed under the supervision of a City inspector.

When required by City regulations, an experienced traffic management subcontractor was retained by Gregg Drilling to provide traffic delineation around drilling operations.

B. Access Restrictions

The JV evaluated all proposed exploration locations prior to site mobilization to determine if any proposed locations were on private or access-restricted land. At locations where access issues were determined, holes were either relocated nearby or removed from the investigation.

Some gaps exist in the numbering scheme at locations where originally planned exploratory holes were deleted.

C. Environmental Regulations

Prior to site mobilization, representatives from both the JV and Gregg Drilling performed a joint site walkover. During this walkover, exploration locations identified within environmentally and/or culturally sensitive areas were either relocated nearby or deleted from the investigation.

Following the site walkover, the JV applied for categorical exemption from the California Environmental Quality Act (CEQA) for each exploratory hole location. CEQA is California statute that requires state and local agencies to identify any significant environmental impacts of applicant's actions and to avoid or mitigate those impacts, if feasible.

CEQA Guidelines §15303(a) and §15304 categorically exempt GIs under the construction of limited new small facilities and minor alterations in the condition of land. CEQA Guideline §15276(a) categorically exempts regional transportation projects. GIs are exempt from CEQA under Guidelines §15306 and §15061(b) (3), provided they do not have a significant effect on the environment.

All applications for categorical exemption were granted for this investigation.

6.2 Cone Penetration Testing Program

CPTs are continuous in situ tests that record geotechnical data through a piezocone that is pushed vertically into the ground at a constant rate of about 20 millimeters per second. The piezocone consists of a conical pointed penetrometer that measures penetration resistance and a cylindrical sleeve that measures frictional resistance. Geotechnical parameters are measured and recorded electronically.

Gregg Drilling completed a total of 44 CPTs from October 10 to October 28, 2011. CPTs were performed following ASTM International (ASTM) test method D 5778. CPTs were completed to depths between 50 and 115 feet, depending on the alignment profile and if refusal was met. A summary of CPT coordinates, elevations, depths, and additional in situ testing, including seismic CPTs (SCPTs) and pore pressure dissipation tests (PPDTs), is presented in Table 6.2-1. A map of test locations is shown in Appendix C.

Table 6.2-1
Summary of CPT Locations, Depths, and In Situ Testing

CPT ID	Northing NAD83 (ft)	Easting NAD83 (ft)	Elevation NAVD88 (ft)	Hand-Auger / Pre-Drill Depths (ft)	Measured Depth to Water (ft)	Total Depth (ft)	In Situ Testing	
							SCPT[1]	PPDT[2]
S0001CPT	2,165,095	6,316,493	296.6	0 to 5	-	80.2		✓
S0002CPT	2,164,037	6,317,351	293.7	0 to 5; 6 to 9	-	50.0		
S0003CPT	2,162,577	6,318,313	287.5	0 to 5	-	80.2		
S0004CPT	2,162,522	6,318,980	289.2	0 to 4.5	-	80.7		
S0005CPT	2,159,881	6,321,692	292.5	0 to 5	-	80.0		✓
S0006CPT	2,159,855	6,320,967	295.1	0 to 15	-	80.1		✓
S0006ACPT	2,158,797	6,322,193	290.1	0 to 5	-	52.5		
S0007CPT	2,157,330	6,323,012	287.1	0 to 5	-	76.4		
S0008CPT	2,156,854	6,324,017	285.3	0 to 5.5; 29 to 36	-	100.2		✓
S0009CPT	2,157,307	6,323,848	286.8	0 to 4.5	-	105.8	✓	✓
S0010CPT	2,156,609	6,324,306	283.4	0 to 4	-	59.2		
S0011CPT	2,156,092	6,324,747	284.4	0 to 6	-	66.6		
S0012CPT	2,155,038	6,324,548	288.9	0 to 4.5	-	102.7	✓	✓
S0013CPT	2,155,267	6,325,407	285.1	0 to 4.5; 9 to 11; 14 to 24;	-	101.2		
S0014CPT	2,154,687	6,325,494	287.6	0 to 5	-	52.2		
S0015CPT	2,153,499	6,326,261	286.1	0 to 5	-	114.7		
S0016CPT	2,152,083	6,327,476	285.4	0 to 5	-	67.3		✓
S0017CPT	2,151,787	6,327,731	286.3	0 to 5	-	80.4		
S0018CPT	2,150,922	6,328,346	286.0	0 to 5	-	80.4		
S0019CPT	2,150,638	6,329,493	289.1	0 to 5	-	81.5	✓	
S0020CPT	2,150,349	6,329,144	289.0	0 to 5	-	80.1		
S0021CPT	2,148,902	6,330,415	292.6	0 to 5	-	80.4		
S0022CPT	2,149,159	6,330,712	293.0	0 to 5	-	80.1		✓

CPT ID	Northing NAD83 (ft)	Easting NAD83 (ft)	Elevation NAVD88 (ft)	Hand-Auger / Pre-Drill Depths (ft)	Measured Depth to Water (ft)	Total Depth (ft)	In Situ Testing	
							SCPT[1]	PPDT[2]
S0023CPT	2,165,095	6,316,493	285.0	0 to 5	94.5	150.6		✓
S0024CPT	2,164,037	6,317,351	285.1	0 to 5	95.0	103.8	✓	✓
S0025CPT	2,162,577	6,318,313	284.7	0 to 5	-	150.6		✓
S0026CPT	2,162,522	6,318,980	283.8	0 to 5	-	64.0		
S0027CPT	2,159,881	6,321,692	286.6	0 to 5	-	80.4		
S0028CPT	2,159,855	6,320,967	286.8	0 to 5	-	45.1		
S0029CPT	2,158,797	6,322,193	287.9	0 to 5	-	80.0		✓
S0030CPT	2,157,330	6,323,012	288.9	0 to 5	-	64.1	✓	
S0031CPT	2,156,854	6,324,017	289.3	0 to 5.5		150.4		✓
S0032CPT	2,157,307	6,323,848	290.1	0 to 4.5	-	90.7		✓
S0033CPT	2,156,609	6,324,306	290.5	0 to 15	-	75.1	✓	
S0034CPT	2,156,092	6,324,747	297.0	0 to 5	-	84.0		
S0034ACPT	2,155,038	6,324,548	303.7	0 to 5	-	95.1	✓	✓
S0035CPT	2,155,267	6,325,407	289.2	0 to 5	61.0	100.2		✓
S0036CPT	2,154,687	6,325,494	288.0	0 to 5; 13 to 22	96.0	100.2		✓
S0037CPT	2,153,499	6,326,261	289.3	0 to 5	31.0	80.0		✓
S0038CPT	2,152,083	6,327,476	287.3	0 to 5	-	50.2		
S0039CPT	2,151,787	6,327,731	290.4	0 to 5	-	50.3		
S0040CPT	2,150,922	6,328,346	289.2	0 to 5	-	55.6		
S0041CPT	2,150,638	6,329,493	293.1	0 to 5	60.9	80.2		✓
S0042CPT	2,150,349	6,329,144	291.5	0 to 5	66.0	80.0		

[1] SCPT: seismic cone penetration test
[2] PPDT: pore pressure dissipation test

CPT locations were spaced at an average of about 1/3 mile apart to develop subsurface soil properties for 30% design.

CPTs have a distinct advantage over boreholes because they provide a continuous profile of tip resistance, friction, and pore pressures generated during penetration while generating minimal investigation waste. The results are most useful in distinguishing changes in stratigraphy, particularly where there may be numerous soil layers with rapid changes with depth. Soft zones embedded within sandy or harder soils can be identified with reasonably good accuracy. Such lenses could be easily missed with conventional boreholes.

CPTs can be used to evaluate soil parameters such as the undrained strength and the strain modulus. Widely used procedures have been developed for using the cone data directly in estimating settlements of footings on sand, load capacities of piles, shear wave velocities, and liquefaction potential. In addition to standard CPT results, specific in situ testing was performed, including the following:

- PPDT to describe the hydrostatic water pressure and permeability of discrete strata, and
- SCPT to define in situ shear wave velocities.

6.2.1 Conventional CPTs

Conventional CPTs were performed to measure penetration resistance, friction resistance, and pore pressure nearly continuously by pushing a piezocone at a constant rate.

6.2.2 Equipment

Two truck-mounted CPT rigs with 30-ton thrust capacity and hydraulic loading systems were on-site for the duration of the CPT investigations. The rigs pushed CPT cones 1 3/4 inches in diameter with a projected base area of 2 1/3 square inches The cone sleeve friction area was 35 square inches. Push rods were 1 3/4 inches in diameter and the grouting rods were 2 inches. The push rods and grouting rods were both 3.3 feet in length.

Each CPT cone was equipped with a porous plastic filter located behind the cone tip that measured pore pressure as the piezocone was advanced and enabled dissipation testing as described in Section 6.2.4.

A. Procedures

Gregg Drilling performed CPT soundings in accordance with ASTM D 5778 procedures. At each test location, they hand-augered holes to a depth of approximately 5 feet to protect unidentified or unknown utilities. Where an obstruction was encountered during hand-augering, the hole was backfilled and a new hole was hand-augered near the original hole.

Once cleared for utilities, the hole was backfilled with sand and the CPT rig was positioned over the hole. The piezocone was pushed using the weight of the CPT rig. Results from the upper 5 feet of disturbed soil were not included in the results, but rather have been indicated with gray shading on the CPT logs.

At some CPT locations, cemented layers were encountered below a depth of 5 feet. At these locations, holes were either (1) pre-drilled using a solid flight augering or (2) “pre-punched” using an oversized steel dummy probe to advance beyond the cemented zone. The CPT locations and depths that required additional pre-drilling and/or pre-punching are noted in Table 6.2-1.

The piezocone was pushed at a constant rate of about 1 inch per second with rod breaks at 3.3-foot intervals. An electronic data acquisition system was used to record measurements of cone tip resistance, sleeve friction, inclination, and pore pressure at about 1 to 2 inch intervals.

CPT soundings were performed to the depths proposed in the Geotechnical Investigation Work Plan (URS/HMM/Arup 2011) or until one of the following criteria was met:

- The hydraulic capacity of the rig was reached,
- The maximum load range for the sensors was reached, or
- The inclination of the push rods exceeded 15 degrees.

When one of these criteria was reached at a depth shallower than planned (unless pre-drilling or pre-punching was performed), the test hole was deemed to have encountered refusal and was terminated.

B. Locations

A summary of CPT coordinates, elevations, depths, and additional in situ testing is presented in Table 6.2-1. A total of 44 CPTs were performed during the investigation. A map of the testing locations is provided in Appendix C.

C. Results

The CPT results provided in Appendix C include plots of tip resistance, friction resistance, friction ratio, pore pressure, and estimated soil behavior type versus depth.

The reported pore pressure values from the CPT logs are not necessarily indicative of the known groundwater conditions, based upon the recorded levels from standpipe piezometers. At some locations, intermittent perched groundwater layers may generate localized pore pressures.

6.2.3 Seismic Cone Penetration Tests

An SCPT was performed in accordance with ASTM D 7400 at the selected CPT locations shown on Table 6.2-1. SCPTs measure compression and shear wave velocities in addition to the standard piezocone parameters. A map showing SCPT locations is provided in Appendix C.

A. Equipment

SCPTs use the same general equipment as described for the conventional CPTs, including the electronic data acquisition system. The primary difference is that the SCPT is fitted with a seismometer to record the arrival of compression and shear waves generated by a surface impact source.

The surface wave was generated by striking a seismic beam fastened to the ground. The beam was struck using either a sledgehammer or an automatic hammer. Before SCPT measurements are recorded, the rods are decoupled from the CPT rig to prevent energy transmission down the rods.

A schematic of the primary elements of the apparatus including the data acquisition system is shown on Figure 6.2-1.

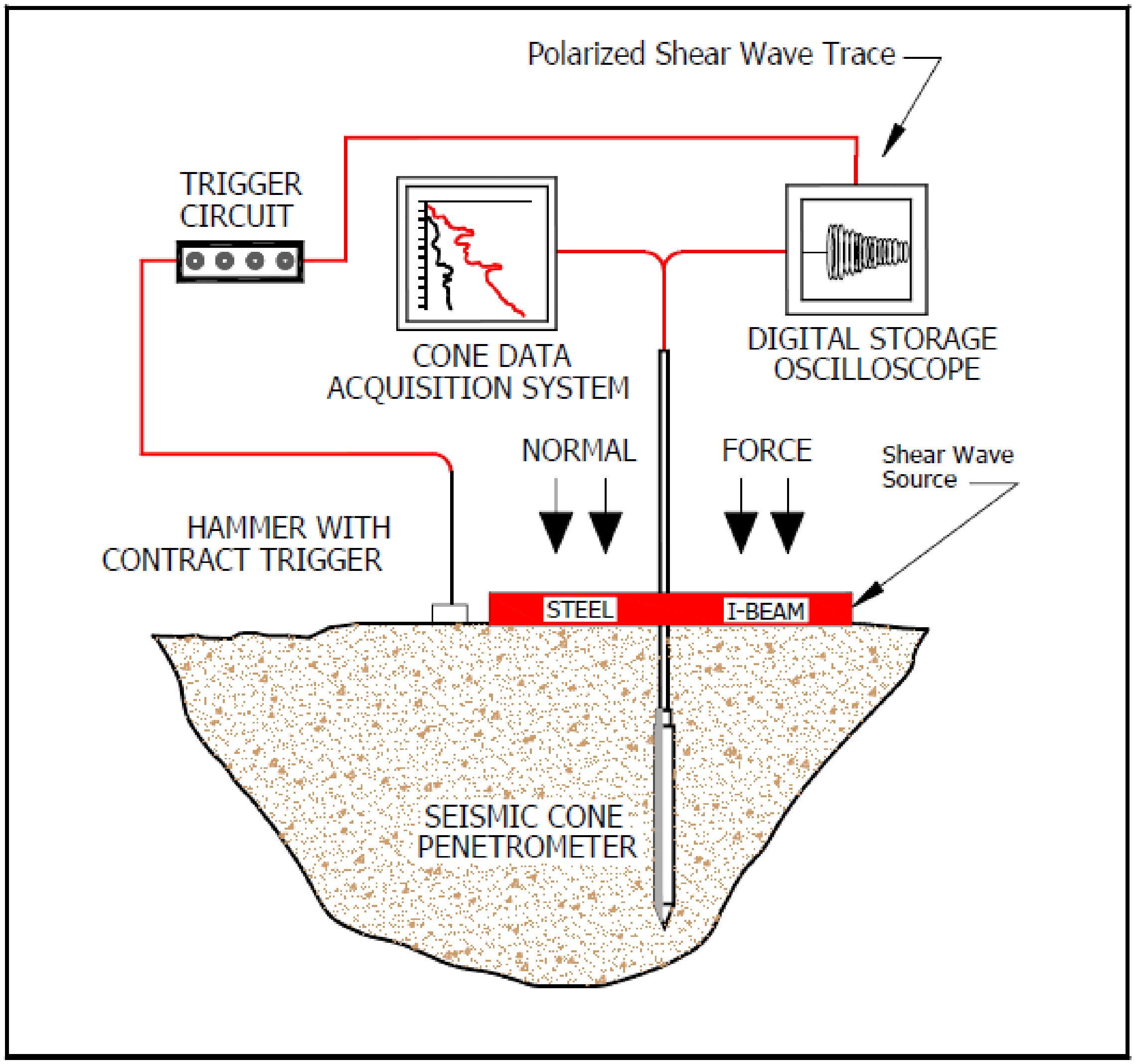

Figure 6.2-1
SCPT Schematic (Gregg Drilling 2011)

B. Procedures

At SCPT locations, a beam was fastened to the ground adjacent to the CPT collar. At each testing depth, the seismic beam column was struck twice on each side of the beam using a sledgehammer or automatic hammers. The striking of the column triggers a record of shear wave velocity in the piezocone at depth. SCPTs were performed at 1-meter intervals.

This measurement was recorded by the electronic data acquisition system and checked for quality by the operator. When necessary, additional hammer strikes were performed at a single depth interval to ensure useable data was retrieved.

C. Locations

A total of six SCPTs were completed during the investigation. The testing locations are shown in Appendix C.

D. Results

Table 6.2-2 shows the results of shear wave velocities averaged over the upper 100 feet (~30 meters) of soil, V_{s30}, from SCPTs. SCPTs that met refusal shallower than 100 feet have been averaged over the depth tested. A graphical profile of measured shear wave velocities is presented in Figure 6.2-2.

Table 6.2-2
Average Shear Wave Velocities from Seismic Cone Penetration Tests

Borehole ID	V_{s30} (ft/sec)	Site Class[1]
S0009CPT	1,461	C
S0012CPT	1,144	D
S0019CPT	1,322[2]	C
S0024CPT	1,197	D
S0030CPT	1,064[3]	D
S0033CPT	1,179[4]	D

[1] As defined in 2006 International Building Code Section 1615.1.5 (ICC 2006)
[2] Averaged over the upper 82 feet of soil
[3] Averaged over the upper 63 feet of soil
[4] Averaged over the upper 72 feet of soil

Based upon the calculated average shear wave velocities obtained from SCPT, the ground profile classifies as between Site Classes C and D by National Earthquake Hazards Reduction Program (NEHRP) definitions (ICC 2006).

Site Class C defines ground profiles with a V_{s30} range of 1,200 to 2,500 feet per second as “very dense soil and soft rock” for design purposes. Site Class D defines ground profiles with V_{s30} ranging from 600 to 1,200 feet per second as “stiff soil” for design purposes.

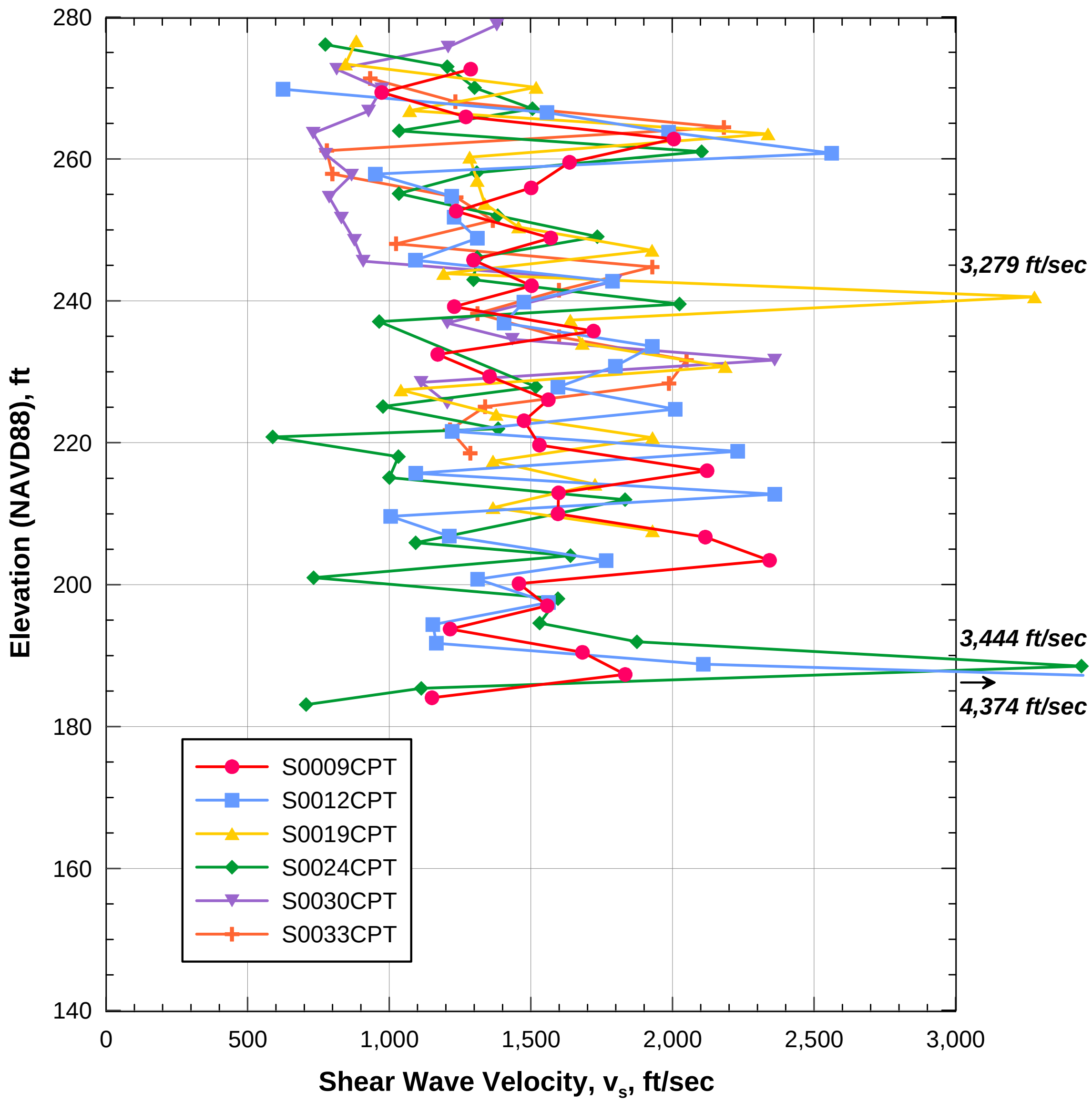

Figure 6.2-2
Shear Wave Velocity Profiles from SCPTs

6.2.4 Pore Pressure Dissipation Testing

PPDTs were performed at select locations during a pause in cone penetration to measure the rate of dissipation of excess pore water pressure generated by the CPT push. The variation of the pore pressure with time was measured via a porous filter located behind the cone tip.

PPDT data can be interpreted to provide estimates of hydrostatic pore water pressure and coefficients of consolidation and permeability at a soil layer of interest. Dissipation tests may be performed rapidly in sands, but may take several hours in high plasticity clays.

The rate of dissipation depends primarily upon soil compressibility and permeability characteristics. However, cone diameter will also influence results as dissipation rate increases with increasing cone diameter.

In order to correctly interpret the equilibrium piezometric pressure, the pore pressure must be monitored until all of the excess pore pressure has dissipated.

A. Equipment

PPDTs used the same equipment described in Section 6.2.1, including an electronic data acquisition system. Piezocones were equipped with a porous plastic filter located behind the cone tip that measured dynamic pore pressure as the piezocone was advanced.

B. Procedures

PPDTs were performed at locations where excess pore pressures were generated during cone penetration, at depths selected by the JV field representative. PPDTs were performed in accordance with ASTM D 5778.

At selected depths, CPT operations were paused to complete the PPDT. Pore pressure was monitored during testing for a maximum of 52 minutes where required to attempt to ensure dissipation of the excess pore pressures. Test durations are summarized in Table 6.2-3.

C. Locations

A total of 19 PPDTs were performed during the investigation. The testing locations are shown in Appendix C.

D. Results

A summary of PPDT results is presented in Table 6.2-3. Graphs showing pore pressure over time for each PPDT are included in Appendix C.

The results of the PPDTs are not necessarily indicative of the known groundwater conditions. Many of the PPDTs were performed at depths shallower than the known depth to groundwater, based upon the recorded levels from standpipe piezometers installed as part of this study. Moreover, many of the test results were not readily conducive to estimating the in situ permeability either because equilibrium pore pressures were not achieved or the pore pressures converged to negative values.

Table 6.2-3 includes estimated Normalized CPT Soil Behavior Type (Robertson 1990) at the elevation of each test for comparative purposes.

Table 6.2-3
Summary of Pore Pressure Dissipation Test Results

CPT ID	Test Depth (ft)	Final Pore Pressure (psi)	Test Duration (sec)	Normalized Soil Behavior Type (SBT_N)[1]
S0001CPT	50.0	1.4	2,390	Gravelly sand to dense sand
S0005CPT	76.8	-4.4	135	Clean sand to silty sand
S0006CPT	80.1	-5.3	135	Clean sand to silty sand
S0008CPT	99.4	-30.8	155	Clean sand to silty sand
S0009CPT	105.0	16.0	165	Gravelly sand to dense sand
	105.8	0.7	1,810	Clean sand to silty sand
S0012CPT	102.2	0.2	1,835	Clean sand to silty sand
S0016CPT	50.4	31.8	530	Clean sand to silty sand
S0022CPT	80.1	44.5	515	Clean sand to silty sand
S0023CPT	113.5	64.1	960	Clean sand to silty sand
	129.6	60.8	3,130	Clean sand to silty sand
	150.6	438.3	30	Clean sand to silty sand
S0024CPT	80.1	14.8	395	Clean sand to silty sand
S0025CPT	118.4	21.8	905	Clean sand to silty sand
S0029CPT	80.1	4.0	585	Clean sand to silty sand
S0031CPT	150.4	28.4	875	Clean sand to silty sand
S0032CPT	90.2	9.2	300	Clean sand to silty sand
S0034ACPT	95.1	1.6	1,135	Clean sand to silty sand
S0035CPT	94.0	12.6	595	Clean sand to silty sand
S0036CPT	100.4	8.9	420	Clean sand to silty sand
S0037CPT	24.1	-0.8	365	Clean sand to silty sand
	80.1	1.1	985	Clean sand to silty sand
S0041CPT	80.2	7.6	625	Clean sand to silty sand

[1] after Robertson (1990)

6.2.5 Cone Penetration Test Completion and Abandonment

All CPTs were backfilled by tremie methods with cement grout in accordance with local permitting agency regulations.

During abandonment, a sacrificial (dummy) tip and hollow rod were pushed back into the original sounding hole to the maximum depth explored. The hollow rod was pulled back to leave the sacrificial tip at the bottom of the hole, and then the hole was backfilled with neat cement grout using the hollow rod as a tremie tube.

A City of Fresno Inspector periodically inspected backfilling of CPTs.

6.3 Exploratory Borehole Program

6.3.1 Overview

A total of 17 exploratory boreholes were completed by Gregg Drilling of Signal Hill, California from October 10 to 28, 2011. These boreholes are as shown in Appendix B. A summary of the all borehole locations is presented in Table 6.3-1. Borehole locations were spaced at an average of about one mile apart to develop subsurface soil properties for 30% design. Boreholes were drilled to a depth of 51.5 to 165 feet, depending on the type of proposed structure.

Table 6.3-1

Summary of Exploratory Borehole Locations, Depths, and In Situ Testing

Borehole ID	Northing, NAD83 (ft)	Easting, NAD83 (ft)	Elevation, NAVD88 (ft)	Continuous Sampling Interval(s) (ft)	Total Depth of Drilling (ft)	In Situ Test Data	
						PS[1]	PZ[2]
S0001R	2,162,577	6,318,315	287.4	5 to 15.5	51.5		
S0002R	2,158,798	6,322,192	290.4	5 to 15.5	81.5		
S0003R	2,157,251	6,323,233	288.0	5 to 15.5	82.0		✓
S0004R	2,156,593	6,324,256	283.7	5 to 15.5; 50 to 56	81.5		
S0005R	2,155,457	6,325,239	285.3	5 to 15.5; 45 to 51	95.0	✓	✓
S0006R	2,154,688	6,325,497	287.6	5 to 15.5; 35 to 41	81.5		
S0007R	2,152,087	6,327,474	285.1	5 to 15.5	81.5		
S0010R	2,150,922	6,328,342	286.1	5 to 15.5	165.0	✓	✓
S0012R	2,148,215	6,330,774	287.6	5 to 15.5	165.0	✓	
S0013AR	2,146,714	6,332,312	286.1	5 to 15.5	150.0		✓
S0014AR	2,143,960	6,334,724	285.4	5 to 15.5	81.5		
S0014R	2,145,253	6,333,705	284.6	5 to 15.5	81.5		
S0015R	2,141,424	6,337,012	286.7	5 to 15.5	51.5		
S0016R	2,138,780	6,338,686	288.8	None	160.0		✓
S0017R	2,136,102	6,340,038	290.5	None	151.5		✓
S0018R	2,134,428	6,340,369	305.8	None	165.0	✓	✓
S0019R	2,125,499	6,341,566	292.5	5 to 15.5	51.5		

[1] PS = P- and S-wave suspension velocity logging
[2] PZ = Standpipe piezometer

6.3.2 Drill Rig and Hammer Types

Drilling was performed primarily using rotary-wash methods with truck-mounted rigs and drilling mud (consisting of bentonite) as the circulating fluid. Hand-augering was performed to a depth of approximately 5 feet at borehole locations to address utility concerns.

At selected borehole locations, a solid flight auger was used prior to rotary-wash drilling to investigate the presence of shallow, perched groundwater.

Samplers were driven with an automatic trip hammer that was calibrated on-site in the first borehole drilled by each rig.

6.3.3 Sampling Methods and Equipment

In accordance with TM 2.9.1 (dated June 2011), the drilling team collected driven soil samples using a 2-inch-outer-diameter SPT sampler. The SPT sampler satisfies the requirements of ASTM D 1586. An interior liner was not used.

In general, samples were obtained at 5-foot intervals to the bottom of each borehole. In selected boreholes, continuous sampling was performed to target particular depths of interest.

At locations where shallow foundations are anticipated, samples were collected continuously from 5 to 15 feet. In boreholes S0004R, S0005R, and S0006R, near the location of the proposed grade separation, 5 feet of continuous sampling was performed at the approximate depth of the proposed structure.

A bulk sample was collected from the upper 5 feet in selected boreholes for compaction testing.

6.3.4 Handheld Field Tests

Soils appropriate for handheld field tests were rarely encountered during this phase of the investigation. When encountered, index strength tests were performed using a torvane and/or hand penetrometer device. Results of handheld field tests have been indicated on the borehole logs presented in Appendix B.

6.3.5 Groundwater-Level Measurements

Groundwater-level measurements were generally not performed during drilling due to the use of drilling mud. Of the 17 boreholes drilled, the only groundwater measured during drilling was in borehole S0001R. A perched groundwater zone was measured at a depth of 13.5 feet.

To monitor groundwater levels, standpipe piezometer observation wells were installed in selected boreholes. Information on the locations, depths, and construction details of standpipe piezometer installations are presented in Section 6.5.

6.3.6 Sample Handling

Samples were preserved and transported in accordance with ASTM D 4220 guidelines. SPT samples were collected in glass sample jars or quart-sized freezer bags. Soil samples were transported periodically to a local storage facility, located at 3636 N Hazel Avenue in Fresno. At this storage facility, samples were reviewed and assigned for laboratory testing. Samples not tested will remain at the storage facility until further notice.

6.3.7 Borehole Completion and Abandonment

All boreholes that were not selected to be converted to piezometers were backfilled with cement grout in accordance with local permitting agency regulations. A City of Fresno inspector periodically inspected the backfilling of boreholes.

Drill cuttings and fluids were initially collected in drums and kept adjacent to the borehole locations until each borehole was completed. Upon completion of each borehole, the drums were transported to the storage facility and consolidated in a single waste container so that the drums could be reused. At the completion of drilling operations, the waste container was characterized for contaminants and then disposed of at an appropriate licensed landfill site.

6.3.8 Borehole Log Organization and Presentation

Upon withdrawal from the borehole, the samplers were cleaned, the sample material classified visually, and the information entered into the field borehole log. The samples were classified using ASTM D 2488 standards supplemented with the Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010). Soil classification procedures are included in Appendix B.

6.3.9 Standard Penetration Tests

Sampling was performed in general accordance with ASTM D 1586. SPT samples were driven using a 140-pound automatic trip hammer falling from a height of 30 inches. Blow counts for the SPT samplers were recorded for each 6-inch interval for an 18-inch drive. The criterion for sampler refusal was considered to be blow counts exceeding 50 for any 6-inch interval.

The combined blow count from the second and third 6-inch intervals is the Standard Penetration Resistance (N-value) and is shown on the borehole logs in Appendix B. It should be noted that the reported N-value has not been corrected for hammer energy, overburden, or field procedures.

6.3.10 Borehole Log Database

Data from the field borehole logs were entered and stored in the geotechnical database program gINT v8. Borehole logs created with gINT are included in Appendix B of this report.

6.3.11 Standard Penetration Test Hammer Energy Calibration

SPT hammer energy calibration testing was performed by Gregg Drilling, on the first borehole drilled by each rotary-wash drill rig. Energy measurements were performed on October 10, 2011, and October 12, 2001, for boreholes S0001R and S0002R, respectively. Measurements were collected using a Model PAK Pile Driving Analyzer.

Hammer energy tests were performed in accordance with ASTM D 4633. Two strain gauges mounted on a 2-foot section of the drill rods measured force, while two piezoresistive accelerometers bolted on the same rod measured acceleration. The gauges were mounted approximately 6 inches from the top of the rod.

The complete reports for each SPT energy calibration are provided as an attachment in Appendix B. The output for each recorded impact of the hammer included the following:

- Blow count in blows per foot
- Maximum rod force
- Maximum rod velocity
- Maximum transferred energy

- Blows per minute
- Energy transferred in percent of maximum

Results from these calibrations indicate an average measured energy transfer of 68 and 88 percent, respectively, for the SPT hammers used in boreholes S0001R and S0002R.

Hammer efficiencies have been noted on each borehole log presented in Appendix B.

6.4 Downhole Geophysical Logging

Site-specific velocity profiles are required to establish site classifications, estimate seismic ground motions, and perform site response analyses for 30% design, as detailed in TM 2.9.6.

Downhole shear wave velocity measurements were completed in selected boreholes using the PS logging method. Logging was performed by GEOVision Geophysical Services, of Corona, California. The summary report prepared by GEOVision is included in Appendix D.

PS logging provides high-resolution measurements (typically spaced at intervals of about 1.5 feet) for the determination of in situ shear and compression wave velocities in deep, uncased boreholes.

The test data provides detailed information regarding the variation of velocities with depth and can accurately differentiate interfaces between layers. The profiles are particularly useful in detecting relatively thin layers of either softer or harder materials that may be interbedded and would be difficult to detect from boreholes alone.

6.4.1 Field Procedures

Downhole seismic surveys were performed in accordance with ASTM D 5753 and D 7400 procedures using method suspension system, manufactured by OYO Corporation. The OYO system uses a 7-meter probe, containing a source and two receivers spaced 1 meter apart, suspended by a cable. The armored 4- or 7-conductor cable serves both to support the probe and to convey data to and from a recording/control device on the surface. The probe is lowered into the borehole to a specified depth, where the source generates a pressure wave in the borehole fluid. The pressure wave is converted to seismic waves (P and S) at the borehole wall. Along the wall at each receiver location, the P and S waves are converted back to pressure waves in the fluid and received by the geophones, which send the data to the recorder on the surface (GeoVision 2012).

Boreholes selected for PS logging were over-drilled 15 feet beyond their sampled depths and flushed with clean water. Measurements were performed in an open hole, below the level of surface casing.

6.4.2 Frequency of Testing

Downhole geophysical logging was performed in four boreholes: S0005R, S0010R, S0012R, and S0018R. A map showing these PS logging locations is provided in Appendix D.

Depths of these boreholes range from 80 to 150 feet. Measurements were recorded in 1.6-foot (about 0.5-meter) Intervals. Table 6.4-1 summarizes PS logging test locations and depths.

Table 6.4-1
Summary of PS Logging

Borehole ID	Date Logged	Northing, NAD83 (ft)	Easting, NAD83 (ft)	Elevation, NAVD88 (ft)	Depth Interval Logged[1]	
					Top Depth (ft)	Bottom Depth (ft)
S0005R	10/17/2011	2,155,457	6,325,239	285.3	6.6	82.0
S0010R	10/19/2011	2,150,922	6,328,342	286.1	6.6	152.6
S0012R	10/25/2011	2,148,215	6,330,774	287.6	1.6	150.9
S0018R	10/28/2011	2,134,428	6,340,369	305.8	26.3	149.3

[1] Logging performed at 1.6-foot intervals between top and bottom depths

6.4.3 Results

Shear wave velocities averaged over the upper 100 feet (~30 meters) of soil, V_{s30}, are presented in Table 6.4-2. A graphical profile of measured shear wave velocities is presented in Figure 6.4-1.

Table 6.4-2
Average Shear Wave Velocities from PS Logging

Borehole ID	V_{s30} (ft/sec)	Site Class[1]
S0005R	1,625[2]	C
S0010R	1,409	C
S0012R	1,566	C
S0018R	1,027	D

[1] As defined in 2006 International Building Code Section 1613.5.5 (ICC 2006)
[2] Averaged over the upper 82 feet of soil

Shear wave velocities were calculated in accordance with the procedures described in the test summary report prepared by GEOVision, provided in Appendix D. This report describes further details of the PS logging method, including test equipment, measurement procedures, and data analysis.

Based upon the calculated average shear wave velocities obtained from PS logging, the ground profile classifies as between Site Classes C and D by NEHRP definitions (ICC 2006).

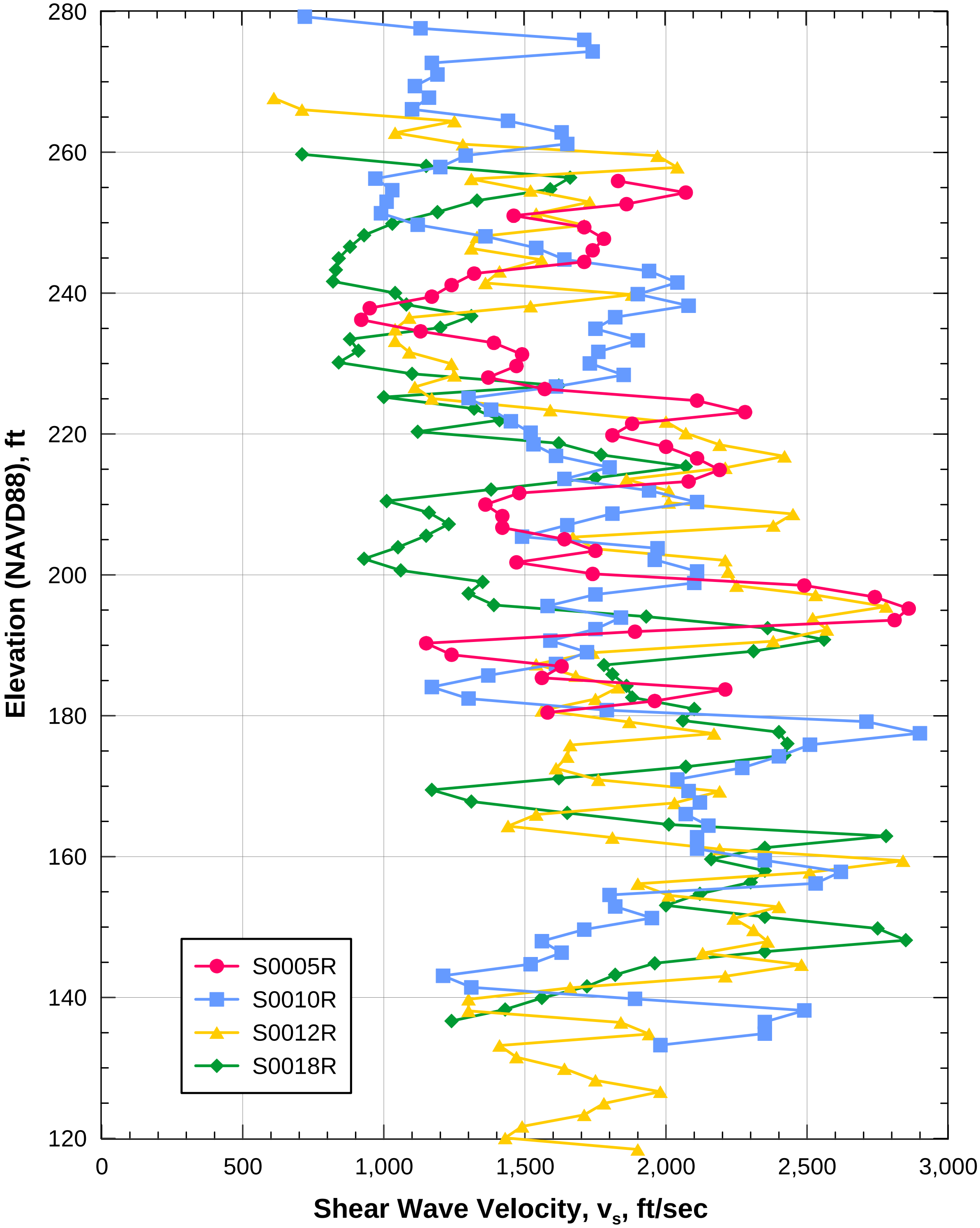

Figure 6.4-1
Shear Wave Velocity Profiles from PS Logging

6.5 Observation Wells

Seven standpipe piezometer observation wells were installed during the GI. During piezometer installation, the depths of the piezometer end cap, sand filter, bentonite seal, and grout, and the length of the slotted screen were recorded. Piezometer installation records are included in the borehole logs in Appendix B and summarized in Table 6.5-1.

Table 6.5-1
Standpipe Piezometer Installation Details

Piezo-meter ID	Date Installed	Northing, NAD83 (ft)	Easting, NAD83 (ft)	Elevation, NAVD88 (ft)	Well Screen Depth[1]		Sand Filter[2]	
					Top Depth (ft)	Bottom Depth (ft)	Top Depth (ft)	Bottom Depth (ft)
S0003R	10/13/2011	2,157,251	6,323,233	288.0	58	78	56	82
S0005R	10/17/2011	2,155,457	6,325,239	285.3	63	83	58	95
S0010R	10/19/2011	2,150,922	6,328,342	286.1	130	150	120	165
S0013AR	10/20/2011	2,146,714	6,332,312	286.1	130	150	128	151.5
S0016R	10/27/2011	2,138,780	6,338,686	288.8	130	150	125	160
S0017R	10/26/2011	2,136,102	6,340,038	290.5	130	150	129	151.5
S0018R	10/31/2011	2,134,428	6,340,369	305.8	130	150	123	151.5

[1] 2-inch-outer-diameter Schedule 40 PVC with 0.020-inch slotted screen
[2] No. 3 Monterey sand

6.5.1 Field Procedures

Standpipe piezometers were installed using the following standard procedures. A typical standpipe piezometer installation is shown in Figure 6.5-1.

- Borehole filled with No. 3 filter sand to desired depth of piezometer end cap
- 2-inch-outer-diameter Schedule 40 PVC solid flush-joint casing with 20-foot length of 0.020-inch slotted screen installed to depth of, at minimum, 2 feet above bentonite chip grout
- No. 3 filter sand tremied into borehole to cover, at minimum, 2 feet above and below screened section of PVC casing
- Bentonite pellet seal placed with, at minimum, 1 feet coverage over No. 3 filter sand pack
- Remaining section of borehole backfilled with cement grout
- Well box installed flush with ground surface

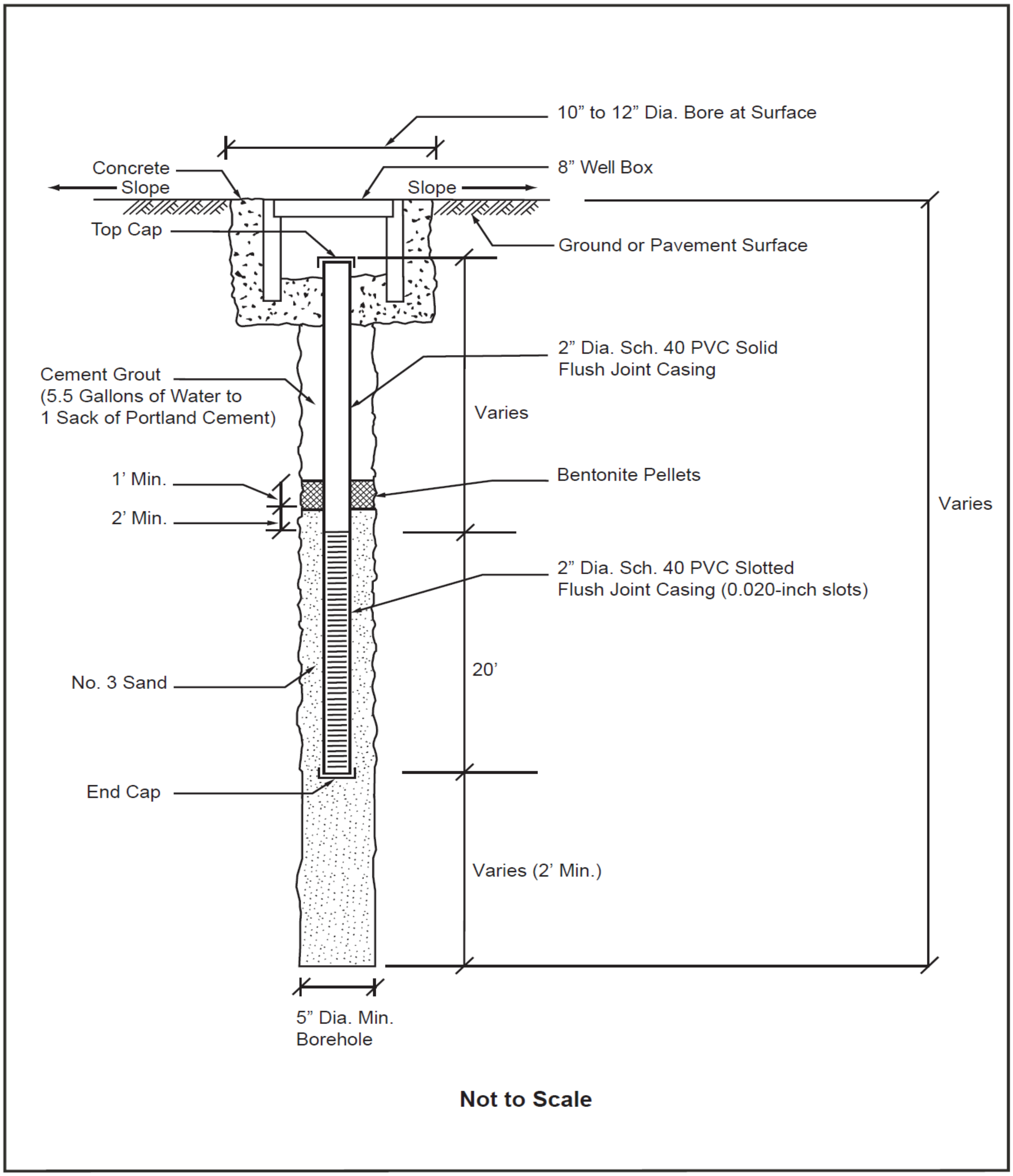

Figure 6.5-1
Typical Standpipe Piezometer Installation

Upon completion of piezometer installations, piezometers were developed in accordance with ASTM D 5521 standards. Piezometer development included bailing and mechanical surging to remove fine-grained materials and drilling fluids from the slotted screen, filter sand pack, and adjacent formation.

Gregg Drilling maintained records of the various operations performed during development, including the type of equipment used, approximate volume of water removed from the piezometers, and static water level before and after development. When possible, at least five well volumes of water were removed from each piezometer.

6.5.2 Frequency of Testing

Piezometers were monitored on a twice monthly basis for the first month after installation to establish a baseline groundwater level. After establishing a baseline, piezometers monitoring continued on a monthly basis to help in understanding long-term groundwater behavior for 30% design and construction cost estimates.

A. Results

Results from the groundwater monitoring program are presented in Table 6.5-2 and Figure 6.5-2.

Table 6.5-2
Groundwater Levels Measured in Standpipe Piezometers

Piezometer ID	S0003R	S0005R	S0010R	S0013AR	S0016R	S0017R	S0018R
Top of Well Box Elevation, NAVD88 (ft)	288.0	285.3	286.1	286.1	288.8	290.5	305.8
Date Read	Measured Depth Groundwater (ft)						
10/13/2011	74.7*						
10/24/2011	Dry*	Dry*	85.7*	92.2*			
11/2/2011	Dry	Dry		93.3	75.2*	78.0*	84.7*
11/9/2011					74.9*	76.7*	
11/10/2011							82.8*
12/2/2011	Dry	Dry	99.6	93.1	75.3	70.5	83.0
1/12/2012	Dry	Dry	98.7	92.0	75.1	70.5	82.5
2/3/12	Dry	Dry	98.4	92.1	75.5	70.8	83.4
*Measured prior to well development							

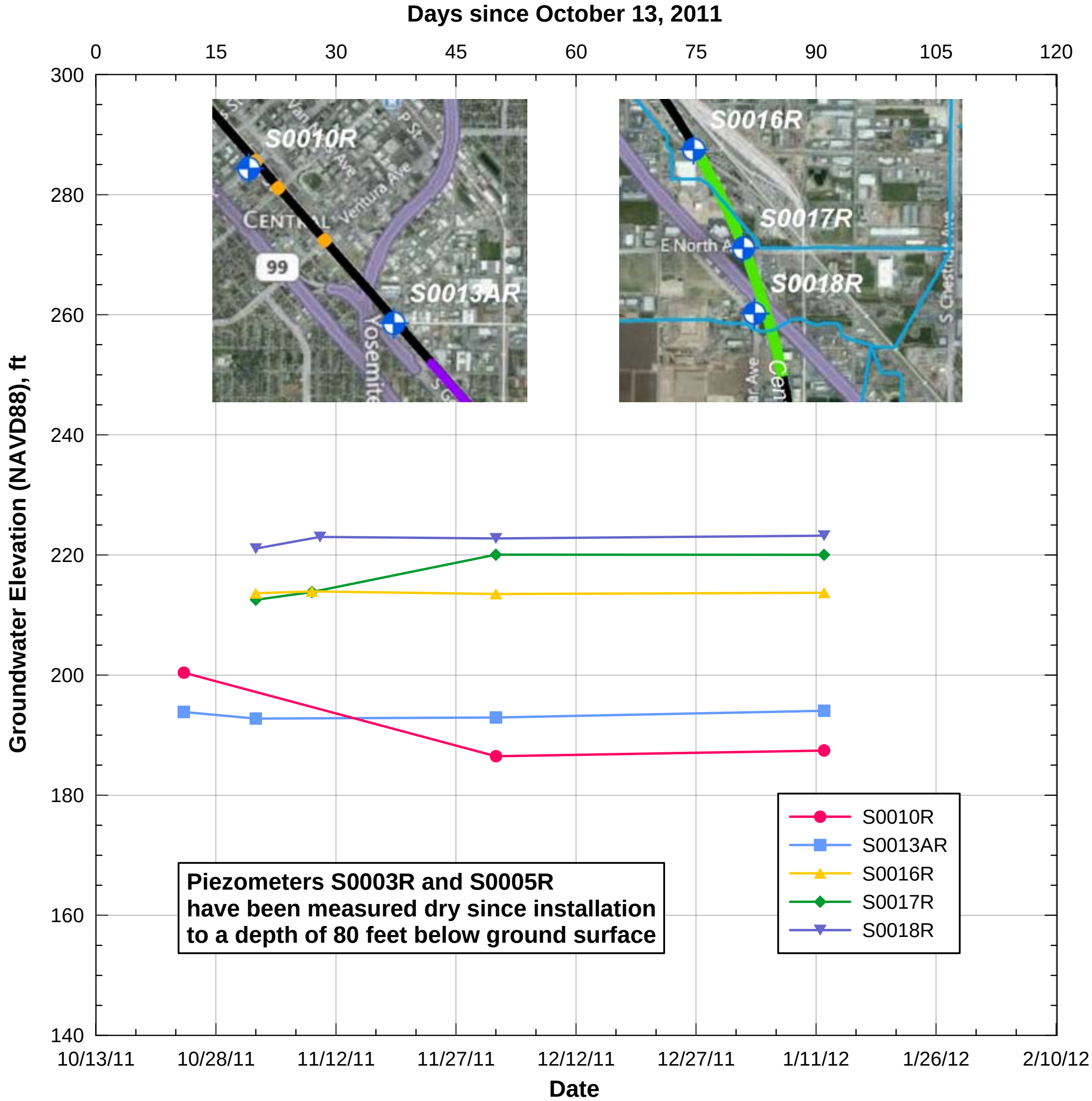

Figure 6.5-2
Groundwater Elevations Measured in Standpipe Piezometers

7.0 Laboratory Investigations

A laboratory test program was completed to provide the necessary data to evaluate the physical and engineering characteristics of soils and groundwater encountered during the ground investigation.

All laboratory testing was performed by Sierra Testing Laboratories, Inc. (Sierra Testing), in El Dorado Hills, California. Soil testing was performed in general accordance with the following ASTM standard test methods:

- ASTM D 422, "Standard Test Method for Particle-Size Analysis of Soils"
- ASTM D 1140, "Test Method for Amount of Material in Soils Finer than the No. 200 Sieve"
- ASTM D 1557, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))"
- ASTM D 1883, "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils"
- ASTM D 2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock"
- ASTM D 2974, "Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils"
- ASTM D 3080, "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions"
- ASTM D 4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
- ASTM D 4327, "Standard Test Method for Anions in Water by Chemically Suppressed Ion Chromatography"
- ASTM G 57, "Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method"

Additionally, groundwater chemistry testing was performed in accordance with the following US Environmental Protection Agency (EPA) and Standard Methods for the Examination of Water and Wastewater (SM) procedures:

- EPA 200.7, "Determination of Metals and Trace Elements in Water and Wastes by Inductively Coupled Plasma-Atomic Spectrometry"
- EPA 300.0, "Inorganic Anions by Ion Chromatography. Official Name: Determination of Inorganic Anions by Ion Chromatography"
- SM 2320B, "Alkalinity"
- SM 2510B, "Conductivity
- SM 4500-H^+B, "pH Value"

Laboratory test results are presented in the following sections. The complete test summary reports by Sierra Testing are included in Appendix E.

7.1 Index Property Testing

Index property testing was performed to provide soil classifications and determine general engineering behavior. The information gathered from index tests may be used to assess the variability of soils and help refine the soil stratigraphy profile along the alignment.

Geotechnical index testing included moisture content, No. 200 sieve wash, hydrometer, grain-size analysis, Atterberg limit, and organic content tests. Table 7.1-1 presents the range of results obtained for each index test performed.

Table 7.1-1
Summary of Results from Index Property Tests

Test	Test Reference	No. of Tests	Range of Values[1]	Mean Value	Standard Deviation
Moisture Content (%)	ASTM D 2216	201	3.9 to 43.5	20.1	7.3
Fines Content (%)	ASTM D 1140	354	0.1 to 97.3	46.3	24.4
Liquid Limit (%)	ASTM D 4318	75	NP to 65	27	9
Plastic Limit (%)	ASTM D 4318	75	NP to 32	20	6
Plasticity Index (%)	ASTM D 4318	75	NP to 49	8	7
Organic Content (%)	ASTM D 2974	43	0 to 5.8	2.3	1.0
[1] NP = Non-plastic					

7.2 Direct Shear Testing

Direct shear tests were performed in accordance with ASTM D 3080 procedures in order to obtain effective-stress soil parameters from remolded specimens.

Direct shear test results are presented in Table 7.2-1.

Table 7.2-1
Summary of Results from Direct Shear Tests on Remolded Specimens

Test Parameter	No. of Tests	Range of Values	Mean Value	Standard Deviation
Effective Cohesion (psf)	51	7 to 1,637	362	377
Effective Friction Angle (deg)	51	24 to 43	37	4

7.3 Compaction Testing

Compaction testing was performed on bulk samples obtained in the fill material to evaluate the compaction characteristics of soils that may be used as fills for embankments, retaining walls, and structural foundations. Increased compaction level generally leads to greater strength and stiffness, and lower settlement under anticipated loading conditions.

Compaction testing was performed using the modified Proctor test method. The modified Proctor test is used to determine the maximum bulk density to which a soil can be compacted given

specified compaction energy. The test may be used to specify the compaction requirements for field control of earthworks.

Compaction test results are presented in Table 7.3-1.

Table 7.3-1
Summary of Results from Modified Proctor Tests

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation
Maximum Dry Unit Weight (pcf)	ASTM D 1557	9	121 to 136.7	128.9	5.8
Optimum Moisture Content (%)		9	6 to 12.2	8.1	2.0

7.4 California Bearing Ratio

California Bearing Ratio (CBR) tests were also performed on bulk samples obtained in the fill material to evaluate its potential strength as a subgrade material. The CBR test measures the response of a compacted soil or aggregate to a bearing pressure. CBR values for each sample were determined at the optimum water content and maximum dry unit weight determined from modified Proctor tests performed at the corresponding test depth.

Compaction test results are presented in Table 7.4-1.

Table 7.4-1
Summary of Results from California Bearing Ratio

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation
CBR	ASTM D 1883	9	13 to 50	30	14

7.5 Soil Corrosion Testing

Corrosion tests were performed on selected samples to evaluate the corrosion potential for buried iron, steel, mortar-coated steel, and reinforced concrete structures. Corrosion testing included pH level, minimum resistivity, and chloride and sulfate concentrations. Corrosion test results are presented in Table 7.5-1.

Table 7.5-1
Summary of Results from Soil Corrosion Tests

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation
Minimum Resistivity (ohm-cm)	ASTM G 57	37	1,130 to 20,900	6,526	4,457
pH	ASTM D 4327	37	6.9 to 8.4	7.6	0.3
Chloride (ppm)	ASTM D 4327	37	6.2 to 124.0	15.0	20.5
Sulfate (ppm)	ASTM D4327	37	0.8 to 273.1	28.8	47.5

For structural elements, Caltrans Corrosion Guidelines (2003) consider a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Resistivity is 1,000 ohm-cm or less
- Chloride concentration is 500 parts per million or greater
- Sulfate concentration is 2,000 parts per million or greater
- pH is 5.5 or less

7.6 Groundwater Chemistry Testing

Groundwater sampling was conducted in three standpipe piezometer to obtain samplers for water chemistry tests. The analytical results may be used to determine the Langelier Saturation and Ryznar Stability Indices, as discussed in Section 5.7. Groundwater chemistry test results are presented in Table 7.6-1.

Table 7.6-1
Summary of Results from Groundwater Chemistry Tests

Test	Test Reference	Borehole ID		
		S0016R	S0017R	S0018R
pH	SM 4500-H^+B	7.51	7.24	7.51
Calcium (mg/L)	EPA 200.7	88	78	47
Bicarbonate Alkalinity as $CaCO_3$ (mg/L)	SM 2320B	280	260	220
Specific Conductance (umhos/cm)	SM 2510B	1,100	860	570
Total Dissolved Solids (mg/L)	SM 2320B	740	580	380
Chloride (mg/L)	EPA 300.0	83	49	23
Sulfate as SO_4 (mg/L)	EPA 300.0	53	110	21

8.0 Surface and Subsurface Conditions along Alignment

8.1 Surface Conditions and Physical Setting

The CP1 alignment corridor spans approximately 9 miles from W Clinton Avenue to just north of E Lincoln Avenue. The alignment travels through primarily industrial, commercial, and suburban residential land within the city limits of Fresno. South of SR 99 the alignment enters primarily rural agricultural, industrial, and residential land.

The alignment follows adjacent to or crosses a number of existing local roads, major highways/freeways, and watercourses. It crosses the Sequoia Kings Canyon Freeway (SR 180) and Golden State Highway (SR 99), and passes under Yosemite Freeway (SR 41). Significant watercourse crossings include the Dry Creek Canal north of SR 180 and the N Central Canal south of E Central Avenue. The alignment also runs adjacent to a detention basin at the intersection of E McKinley Avenue and N Golden State Boulevard, and at W Belmont Avenue.

The ground level along the CP1 study area is relatively flat, with a general downward gradient to the west-southwest. Ground surface elevations measured at exploratory hole locations ranged from about 283 to about 306 feet (NAVD88). Surface elevations are determined principally by the gentle slope of the vast alluvial fans extending from the Sierra Nevada in the east to the center of the SJV.

8.2 Generalized Subsurface Conditions

Subsurface soils have been characterized into two separate layers: (1) Existing Fill and (2) Alluvial Fan. The Alluvial Fan stratum is assumed to include the Modesto and Riverbank formations and Sand Dunes. The Turlock formation was not encountered during the field exploration and is not anticipated to be encountered during construction. A distinction was not made between the Modesto, Riverbank, and Sand Dunes because the investigation did not identify a discernable difference between their composition and engineering properties.

The following sections describe the subsurface ground conditions encountered, including groundwater conditions and evidence of soil contamination.

8.2.1 Existing Fill

Existing Fill encountered during the GI varied from 1 to 7 feet in thickness. Existing Fill consists of silty sand (SM), sand with silt (SP-SM), and sandy silt (ML), and contains varying amounts of fine gravel.

Historical records describing how Existing Fill was placed and compacted were not found during our investigation. The largest Existing Fill fragment encountered during the investigation was less than 1 inch in greatest dimension. However, the nature of drilling and sampling methods used and borehole spacing makes it difficult to quantify the maximum size of fragments in Existing Fill.

Existing Fill included surface pavements consisting of asphalt concrete, concrete, and aggregate base. Where encountered, existing asphalt concrete varied from 4 to 8 inches in thickness, aggregate base from 0 to 9 inches, and concrete for road gutter about 12 inches at boring S0004R. Ceramic and glass debris were present in Existing Fill encountered in borehole S0001R.

Few laboratory tests were performed on Existing Fill because the bulk samples collected were highly disturbed and were taken from drilling cuttings. Laboratory tests performed included

Modified Proctor Compaction, CBR, moisture content, and fines content. These tests were performed to evaluate pavement design and earthwork considerations.

Modified Proctor test results in the fill show maximum dry unit weights between 121.0 and 136.7 pounds per cubic foot, and optimum moisture contents ranging from 6.0% to 12.2%. CBR values ranged from 13 to 50.

Grain-size analyses in the bulk samples tested indicate trace fine gravel (less than 1% by weight).

8.2.2 Alluvial Fan

Alluvial Fan (Qc, Qf, and Qs) was present beneath Existing Fill to the maximum depth explored. It consists of interbedded layers of poorly graded sand and silt, with varying amounts of coarse and fine grained particles. Interlayers of this unit are classified as poorly graded sand (SP), sand with silt (SP-SM), silty sand (SM), clayey sand (SC), clay (CL), silty clay (CL-ML), sandy silt (ML), silt with sand (ML) and silt (ML).

A histogram of USCS distributions in the Alluvial Fan layer is shown in Figure 8.2-1

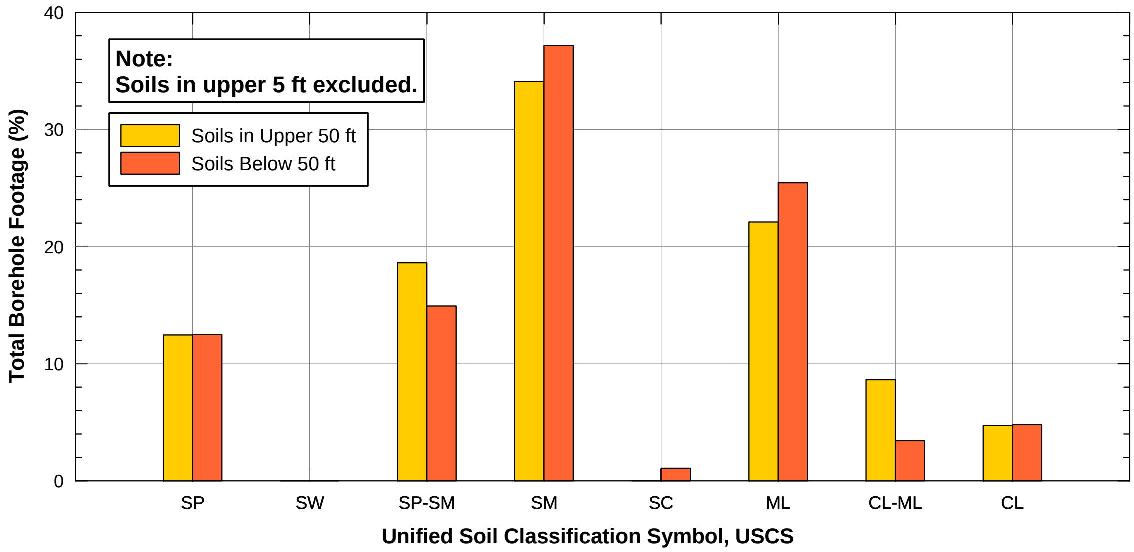

Figure 8.2-1
Unified Soil Classification System (USCS) Distribution for Alluvial Fan

A histogram of Normalized CPT Soil Behavior Type (SBT_N) distributions in the Alluvial Fan layer is shown in Figure 8.2-2. The SBT_N classification (Robertson 1990) may be used as a guide to predict soil type based on cone penetration resistance and sleeve friction.

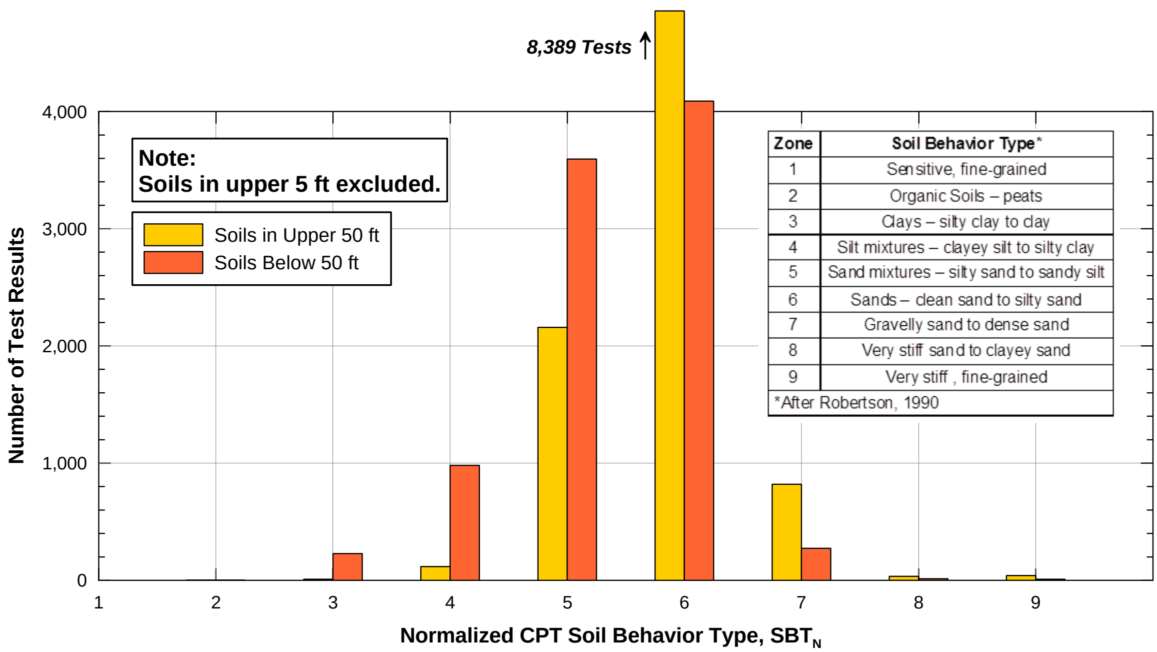

Figure 8.2-2
Normalized CPT Soil Behavior Type (SBTN) Distribution for Alluvial Fan

Hardpan soil was present within the Alluvial Fan layer at variable depths. Boreholes drilled during the investigation encountered hardpan layers varying from 1 to 5 feet in thickness are present between 5 and 15 feet BGS. Where sampled, hardpan was hard and very dense and consists of sandy silt (ML), silt (ML), silt with sand (ML), silty sand (SM), and sand with silt (SP-SM).

SPT N-values measured in hardpan were greater than 50 blows per foot in ML soils and greater than 100 blows per foot in SM and SP-SM soils.

A profile of SPT N-values measured in each borehole is presented in Figure 8.2-3. The SPT N-values shown in Figure 8.2-1 have not been corrected for energy and field procedures. Uncorrected SPT N-values range from 6 to values greater than 50 blows per 6 inches (i.e., sampler refusal).

A cross-section profile of the subsurface stratigraphy is presented in Figures 8.3-4 and 8.3-5.

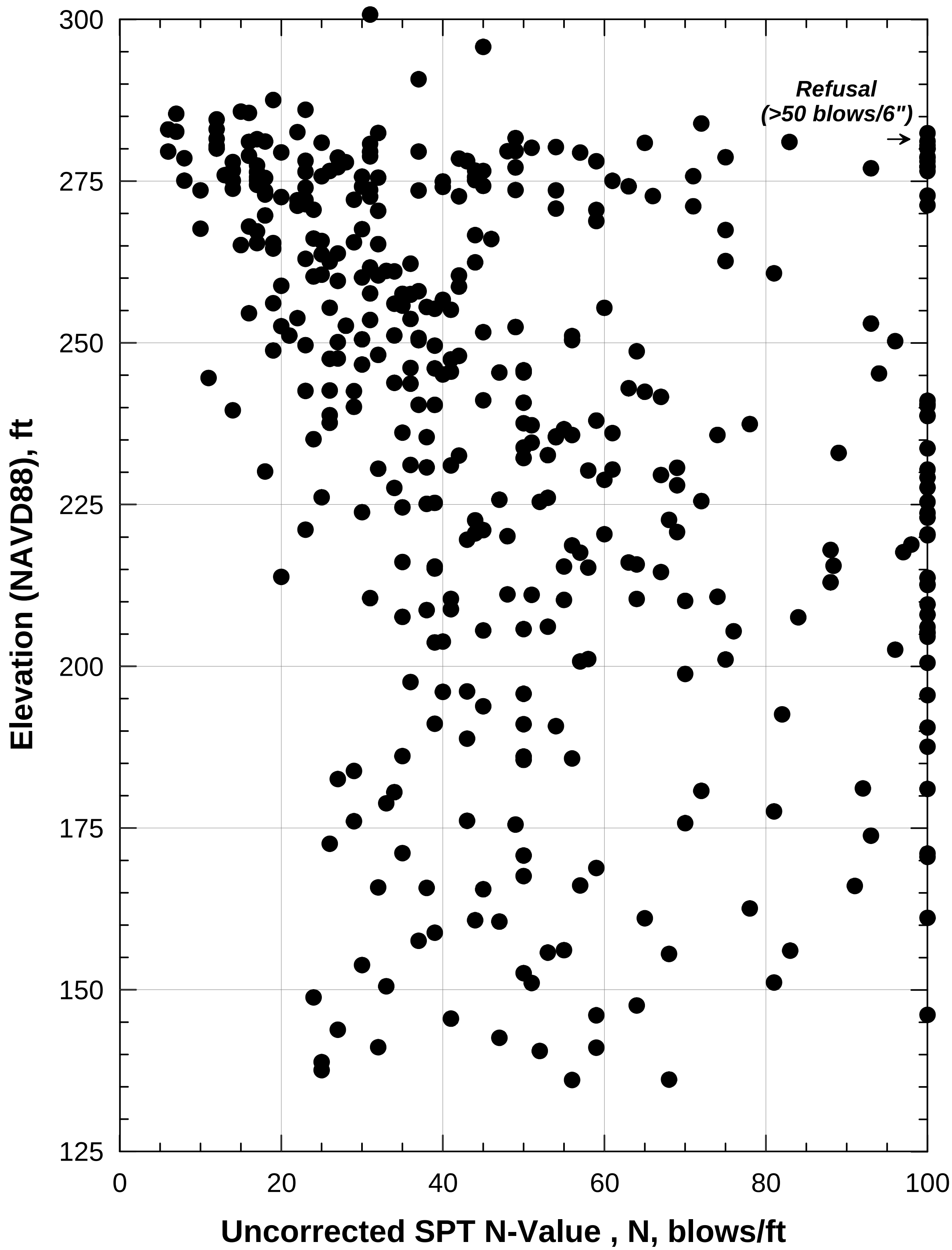

Figure 8.2-3
Profile of SPT N-Values

FIGURE 8.2-4
SUB SURFACE STRATIGRAPHY ALONG STUDY AREA

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FIGURE 8.2-5
SUB SURFACE STRATIGRAPHY ALONG STUDY AREA

8.2.3 Contaminated Soil

Evidence of soil contamination was noted in three exploratory holes. The locations where contaminated soils were encountered are presented in Table 8.2-1.

Table 8.2-1
Evidence of Contaminated Soils Encountered in Exploratory Holes

Hole ID	Description of Contamination
S0012R	Strong hydrocarbon odor and visual presence of contamination in SPT samples obtained at depths of 25 and 30 feet; no laboratory testing was performed to confirm contaminant type or concentration.
S0014AR	Pinkish-red contaminant visually identified in SPT samples obtained at depths of 11 and 12.5 feet; no laboratory testing was performed to confirm contaminant type or concentration.
S0019CPT	Hydrocarbon odor present in CPT hole in upper 20 feet

No other evidence of soil contamination was noted in any other exploratory holes during the investigation. However, contaminated soil could exist at other locations along the alignment.

Current and historical land use in the vicinity indicates man-made hazardous materials are likely to exist throughout the areas in and around the CP1 alignment. Hazardous materials associated with man-made contamination can include petroleum hydrocarbons, volatile organic compounds, semivolatile organic compounds, pesticides, PCBs, and metals. These contaminants are usually associated with former agricultural, industrial, and/or commercial land uses. Aerially deposited lead is common in soil along shoulders of major thoroughfares from past leaded fuel vehicle emissions.

Railroads have historically used lead arsenate or other arsenic compounds as pesticide and herbicide, as well as using chlorinated pesticides. Lead may also be present as lead-based paint debris or as aerially-deposited lead. PCBs were historically used in railroad electrical equipment.

8.2.4 Groundwater Conditions

Monitoring of groundwater conditions was conducted in seven standpipe piezometers installed during the GI. Groundwater-level measurements were generally not performed in boreholes during drilling due to the use of drilling fluid.

Results from piezometer measurements indicate the groundwater table is below 80 feet BGS in boreholes S0003R and S0005R, and between 70 and 100 feet in boreholes S0010R, S0013AR, S0016R, S0017R, and S0018R. To date, groundwater levels measured in standpipe piezometers have not been subject to significant seasonal fluctuation.

Groundwater level measurements were also recorded in several CPTs. Measurements recorded from S0023ACPT, S0035CPT, S0036CPT, and S0041CPT indicate depths to groundwater between 61 and 95 feet.

A. Perched Groundwater

Despite the lack of perched groundwater data from the exploration, there is a potential for perched groundwater to exist near existing watercourses, including detention basins and irrigation canals. Local perched water may also occur above hardpan layers.

Of the 17 boreholes drilled, the only perched groundwater measured during drilling occurred in borehole S0001R at a depth of 13.5 feet. This borehole was located near the detention basin at the intersection of W McKinley and N Weber Avenues.

Borehole S0004R was drilled using a solid flight auger to a depth of 21.5 feet to determine if perched groundwater conditions were present near detention basin at W Belmont Avenue. No groundwater was encountered to this depth.

Perched groundwater was measured at a depth 31 feet in S0037CPT. This CPT was performed adjacent to the Central Canal.

9.0 Limitations and Further Information

The JV warrants that its services were performed within the limits prescribed by our client for the project, in a manner consistent with the level of care and skill ordinarily exercised by members of the same profession currently practicing in the same locality under similar circumstances. No other warranty or representation, either expressed or implied, is included or intended hereunder.

This report contains results of a preliminary geotechnical study to support 30% design. The geotechnical data presented in this report were collected based on the indicated project criteria and are intended only for the purposes, site location, and project described in this report. Conclusions drawn from the data presented herein are subject to change based on (1) future explorations to be performed by the Contractor, (2) when additional information on subsurface conditions becomes publically available, and/or (3) key features of the project are changed during design.

The JV cannot be held responsible for interpretations, professional opinions, or advice given by others with regard to any geotechnical data presented in this report.

Additional explorations and/or analyses will be required to develop and prepare the GBR for Construction, which provides the basis for final design and construction. A geotechnical study for the project should reassess the geotechnical considerations and preliminary criteria presented in this report, and include additional geotechnical explorations, laboratory testing, and engineering analyses deemed necessary by the design-build contractor's geotechnical engineer of record to provide geotechnical design parameters for the final design of the proposed structures.

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